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Evaluation of the Effect of Binder Grades and Modifiers on Rutting Performance

By

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DECLARATION

I, the undersigned, declare that this thesis is my original work and has not been presented for a degree in any other University, and that all sources of materials used for the thesis have been duly acknowledged.

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ABSTRACT

Rutting is recognized to be the major distress mechanism in flexible pavements. Rutting is caused by the accumulation of permanent deformation in all or some of the layers in the pavement structure. The accumulation of permanent deformation in the asphalt surfacing layer is now recognized to be the major component of rutting in flexible pavement.

Rutting especially is a major problem for roads with a relatively steeper gradient and high pavement temperature like Goha Tsiyon – Dejen Road and Adama - Metehara Road of Ethiopia. The binder grade used was 60/70 according to ERA design manual and no modifier was used.

This study, thus, attempts to evaluate the rutting potential of the asphalt concrete mixes prepared with different binder grades and modifiers. The different binders were compared by making specimens of the mix design. A wheel Tracker called Quan Zi Dong Che Zhe Shi Yan Yi, a Chinese device, was used to evaluate rutting performance of the compacted samples. The testing and statistical analysis of the rut depths provides evidence that the mix with 40/50 binder grade performs better than 60/70 and the use of modifier (NES) in the asphalt concrete mix provide mixes with superior performance.

Key Words: Binder grade, Modifier, Rutting, Quan Zi Dong Che Zhe Shi Yan Yi, Statistical analysis

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Abbreviation	Meaning
AASHTO	American Association of State Highway and Transportation Officials
AC	Asphalt Cement
ASTM	American Society for Testing Materials
CZCXY	Che Zhe Cheng Xing Yi
Gmb	Bulk Specific Gravity
Gmm	Maximum Specific Gravity
HMA	Hot Mix Asphalt
MTD	Maximum Theoretical Density
PMA	Polymer Modified Asphalt
Ps	Aggregate as Percent of Total Weight
QZDCZSYY	Quan Zi Dong Che Zhe Shi Yan Yi
SSD	Saturated Surface Dry
SBS	Styrene-Butadiene-Styrene
SHRP	Strategic Highway Research Program
SuperPave	Superior Performing Pavements
Va	Air Void
VFA	Voids Filled with Asphalt
VMA	Voids in Mineral Aggregate
WT	Wheel Tracker

CHAPTER 1 INTRODUCTION

1.1 BACKGROUND

Road network of suitable quality and safety plays vital role for the economic development of a country. However, during the past several years many roads experienced problems with amount and severity of permanent deformation in hot mix asphalt pavements. This problem with permanent deformation, or rutting, was attributed to an increase in truck tire pressures, axle loads, and volume of traffic (Brown *et al*, 1992).

Permanent deformation in the form of rutting is one of the most important distress mechanisms in asphalt pavements. With increase in truck tire pressure in recent years, rutting has become the dominant mode of flexible pavement failure. Pavement rutting, which results in a distorted pavement surface, is primarily caused by the accumulation of permanent deformation in all or a portion of the layers in the pavement structure. Longitudinal variability in the magnitude of rutting causes roughness. Water may become trapped in ruts resulting in a reduced skid resistance, increased potential for hydroplaning and spray that reduces visibility. Progression of rutting can lead to cracking and eventually to complete disintegration or failure. Rutting accounts for a significant portion of maintenance and associated costs in both main highways and secondary roads.

One of the many points that has to be considered to alleviate rutting is the binder grades and modifiers to be used in asphalt concrete mix. Especially to avoid premature rutting, selection of the right binder grade and modifier is essential. In Ethiopia we are still using the penetration grading system of binder selection. However, the Strategic Highway Research Program of USA had produced a new system called performance grading, PG, of the asphalt binder which is now widely implemented by the highway community. The PG specification incorporates the entire temperature range the binders experience both during the construction and while in service. This binder specification was specifically designed to address the binder's role with respect to three types of pavement distress: rutting, fatigue, and low temperature cracking.

Rutting is a major problem for some roads in Ethiopia like Goha Tsiyon – Dejen Road, Adama – Metehara Road etc. In Goha Tsiyon – Dejen Road Rehabilitation Project, rutting was observed on considerable length of the project road after two months of the asphalt concrete construction according to the project completion report by the consultant of the project. The binder grade used was 60/70 according to ERA Design Manual. The objective of this research is to evaluate the asphalt concrete mix produced with 60/70 binder grade (used at the mentioned project) and compare it to the expected performance of asphalt concrete mixes with 40/50 without modifier and 40/50 & 60/70 with the addition of modifiers.

1.2 PROBLEM STATEMENT

In recent decades, pavement engineers have been challenged to use conventional methods to design cost effective pavements that are to withstand unconventional wheel loads and tire pressures. Large stone mixes are becoming a popular means for reducing rutting in flexible pavements. Heavy concentration of aggregate interlock in large stone mixes allows for efficient dissipation of compressive and shear stresses that are otherwise known to be responsible for rutting and shoving in flexible pavements (Mahboub *et al*, 1990). Some polymer-modified asphalt cements are being used in asphalt concrete pavements because of their role in reducing several types of pavement distress and enhancing pavement performance according to Khattak (Zaniewski *et al*, 2003).

Asphalt surface construction for Goha Tsion – Dejen Road Rehabilibtaion project was commenced after the laboratory test procedures to come up with the best job mix and necessary site preparation works. However, after two months of construction on a considerable length of the project road, rutting was noticed on the pavement according to the results of the investigation by the consultant of the project.

Rutting reduces safety, comfort and level of service (LOS) of the road. It also has significant cost implication due to maintenance or reconstruction. Significant rutting can lead to major structural failure and hydroplaning, which is a safety hazard. To alleviate such kind of problem, evaluation of the rutting potential of the asphalt mix using rutting measuring equipment before commencement of Asphalt surface construction is very important. This study compares rutting potential of the asphalt concrete using different asphalt binder grades and modifiers for the prevailing temperature and traffic load.

Figure 1.1 Condition of Goha Tsion – Dejen Road

1.3 OBJECTIVE

General Objective

The general objective of this research is to evaluate the rutting potential of the asphalt concrete mixes. The evaluations of the rutting potential of the asphalt concrete mix help us choose a better asphalt concrete mix which is less susceptible to rutting.

Specific Objectives

The specific objectives of this study are:

- To evaluate the effect of binder grades on rutting potential of the asphalt concrete mixes.

- To evaluate the effect of modifier on the rutting potential of the asphalt concrete mixes
- To evaluate the effect of binder grades and modifier on the marshal properties of the asphalt concrete mixes.

1.4 SCOPE

The research reported herein was focused on evaluation of binder grades and modifiers on rutting performance. The study was carried out considering and comparing different binder grades and modifiers which help in selecting the right binder type for the HMA pavement of specific area. The material selected for this study i.e. aggregate and bitumen were collected from China Communication and Construction Company LTD, CCCC and the modifier is supplied by SDHS Qingchuan Road Materials Development Co. Ltd in association with Rosnar Holding Co. Ltd. The laboratory of Addis Ababa - Adama Toll Motorway project was used for laboratory works of the research.

The mixtures were prepared using each type of binder grades without and with modifier. The results produced in this research were based on Marshal Mix Design Method. A detailed laboratory testing was performed on samples of the asphalt concrete mixture from the different binder grades and modifier. The data were analyzed to determine material and mixture for construction of rut resistant HMA pavements.

CHAPTER 2 LITERATURE REVIEW

2.1 INTRODUCTION

Asphalt pavements typically provide excellent performance and value. They are smooth and durable. They do not require long construction times and they are easy to maintain resulting in minimal traffic delays. Asphalt surfaced roads subjected to heavy traffic in hot climates may experience early failures in the form of rutting. The rutting failures are the result of heavy truckloads with high tire pressures and high pavement temperatures (Zaniewski *et al*, 2003).

Careful selection of asphalt binder and aggregate combination will help in providing optimum performing Hot Mix Asphalt, HMA, pavements. The use of Performance Graded binder system has the advantage of the binder being selected based on the climate in which it will serve. The aggregate structure used must be capable of carrying the load and developing a high degree of stone-to-stone interlock that will resist shear. In addition to materials selection, the mix design procedure is crucial in achieving desired performance.

The Penetration grade binder tests that were performed on the asphalt binders, rutting and modified asphalt binders are discussed in this chapter along with the rut measuring equipment.

2.2 GRADE OF BITUMEN FOR FLEXIBLE PAVEMENTS

Currently, majority of Ethiopian roads are flexible pavements, the ones having bituminous layer. Flexible pavement are preferred over cement concrete roads as they have a great advantage that these can be strengthened and improved in stages with the growth of traffic. Another major advantage of these roads is that their surfaces can be milled and recycled for rehabilitation. The flexible pavements are less expensive also with regard to initial investment and maintenance.

2.2.1 Significance of Properties of Bitumen

The durability and the long term satisfactory performance of pavements are always influenced and affected to a greater extent by the employed pavement ingredient materials and their inherent properties. The upper layers of a pavement structure are vital in taking care of stress alleviation and protecting the structure. They are conceived as layers of superior quality materials in the structure and are constructed accordingly.

In bituminous pavements, stone aggregates and bituminous binder are the key ingredients and hence are desired to be of good quality, making their selection an important task. The applicability and adhesive properties of bitumen along with the proper proportioning with stone aggregates is the basic requirement to make workable layer mixes. Asphaltic bitumen is obtained by refining the petroleum crude. It is the costliest and a very important component of the bituminous mix.

It is very much pertinent to consider the properties of bituminous binders and the bitumen content in a mix while attempting for enhancing the performance characteristics of bituminous mixes. The construction sector is interested in using a right type of bitumen for obtaining durable pavements with longevity of more than 10 years.

2.2.2 Bitumen Tests

There are age old conventional tests as well as new generation tests to help in framing the specifications for bituminous binders, such as:

1. Conventional tests used for bitumen characterization
   - Penetration, ductility, softening point, flash point, Fraass breaking point…..
2. Ageing characteristics:
   - Thin Film Oven Test, Rolling Thin Film Oven Test, Pressure Aging Vessel…
3. Rheological tests:
   - Bending Beam Rheometer, Direct tension Test, Dynamic Shear Rheometer…..

It is imminent to select an array of above tests to assist in the selection of right quality of bitumen for pavement applications.

2.2.3 Global Practices in Right Grade of Bitumen Selection

Stiffness of bitumen is the ultimate phenomenon for bitumen, while chewing in mouth was the earliest mode of testing bitumen; hence the test temperature at that time was 37°C (which is the temperature of human body). In 1903, American Society for Testing of Materials (ASTM) adopted the grading of bitumen through penetration testing at 25°C. The lower penetration value indicated the harder bitumen, while the higher penetration value indicated the softer bitumen. The penetration based grading system continued until 1970 and the same continues even today in many countries including Ethiopia. Thereafter in order to address various performance related problems such as rutting at high pavement temperatures, viscosity based grading system (Viscosity at 60°C) was introduced in US in the year 1970.

In 1987, the Strategic Highway Research Program (SHRP) began developing a new system for specifying asphalt materials. The product of the SHRP asphalt research program was a materials selection, testing, and evaluation design system called Superpave. The Superpave binder specification is performance based, resulting in the classification of performance-graded (PG) binders. It specifies binders on the basis of the climate and pavement temperatures in which the binder is expected to serve. Performance graded (PG) binders are graded such as PG 64-16. The first number, 64, is often called the "high temperature grade." This means that the binder would possess adequate physical properties at least up to 64°C. This would be the high pavement temperature corresponding to the climate in which the binder is actually expected to serve. Likewise, the second number (-16) is often called the "low temperature grade" and means that the binder would possess adequate physical properties in pavements at least down to -16° C. Therefore, PG-64-16 bitumen is suitable for a project location, where average 7 days maximum pavement temperature is as high as 64°C and the minimum pavement temperature is as low as -16°C. (Nagabhushana, 2009)

2.2.4 Ethiopian Scenario and Associated Problems

In Ethiopia, the bitumen grading is practiced on the basis of penetration test, which is conducted at a temperature of 25°C. The empirical penetration test was developed over 100 years ago.

The most common problem in the performance of bituminous concrete roads (50 mm or thicker) throughout the world including Ethiopia is rutting during hot summer. The bitumen becomes soft in the 60 to 70°C temperature range (road surface temperature on a hot summer day) and starts to push and shove under loaded truck tyres leading to rutting and corrugations in the wheel tracks of the roadway.

Bitumen processed from different petroleum crude sources and/or refining processes may have the same penetration grade at 25°C but may exhibit significantly different hardness in the 60-70°C temperature range. Those which are very soft (low viscosity) are more prone to rutting and corrugations compared to those which are not as soft (high viscosity). Therefore, it is quite obvious that the consistency (viscosity) of the paving bitumen at high temperature (such as 60°C) is to be invariably determined to know if it is likely to cause rutting or not. Because of this problem, a requirement to test and grade the bitumen at 60°C (and not at 25°C) through viscosity test was implemented in U.S. during 1970s. The viscosity of the bitumen is measured in poises at 60°C using a simple viscometer. Various viscosity grades of asphalt cement (bitumen) were evolved as AC-30 (asphalt cement-30) Grade, AC-20 Grade and AC-10 Grade. Later, the performance-graded (PG) binders are developed which are specified based of the climate and pavement temperatures in which the binder is expected to serve. (Nagabhushana, 2009)

Although the viscosity grading and performance grading system are developed after penetration grading system, Ethiopia is still using the penetration grading system. The widely used binder types in Ethiopia are 80/100 and 60/70 penetration grade.

2.3 PENETRATION GRADE BINDER TESTS

Penetration grade bitumen is specified by the penetration test and softening point test. Designation is by penetration range only, e.g. 40/60 penetration bitumen has a penetration that ranges from 40 to 60 inclusive and a softening point of 48 ^{o}C to 56 ^{o}C. The unit of penetration is given as decimillimeter (dmm). There are three common types of penetration grade bitumen for asphalt concrete mixes i.e. 40/50, 60/70 and 80/100. However, for surface dressing 150/200 is used.

2.3.1 The Penetration Test

The consistency of penetration grade bitumen is measured by penetration test. In this test, a needle of specified dimensions is allowed to penetrate a sample of bitumen, under a known load (100gm), at a fixed temperature (25 ^{o} C), for a known time (5second).

The penetration is defined as the vertical distance travelled by the needle into the bitumen. It is measured in tenth of a millimeter (decimillimeter,dmm). The lower the value of penetration, the harder the bitumen. Conversely, the higher the value of penetration, the softer the bitumen. It is essential that the test methods are followed precisely as even a slight variation can cause large difference in the result.

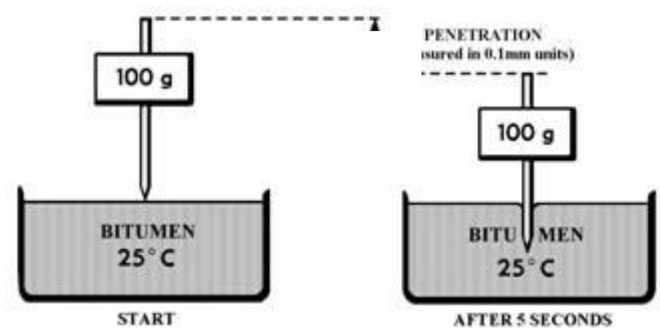

Figure 2.1: Standard penetration test

2.3.2 The softening point test

The consistency of penetration grade bitumen can also be measured by determining its softening point. In this test, a steel ball (weight 3.5g) is placed on a sample of bitumen contained in a brass ring that is then suspended in a water or glycerin bath.

Water is used for bitumen with a softening point of 80 ^{o} C or below and glycerin is used for softening points greater than 80 ^{o} C. The bath temperature is raised at 5 ^{o} C per minute, the bitumen softens and eventually deforms slowly with the ball through the ring. At the moment the bitumen and steel ball touch a base plate 25mm below the ring, the temperature of the water is recorded. The test is performed twice and the mean of the two measured temperature is reported to the nearest 0.2 ^{o} C. If the difference between the results exceeds 1 ^{o} C, the test must be repeated. The softening point of the bitumen represents an equi-viscous temperature.

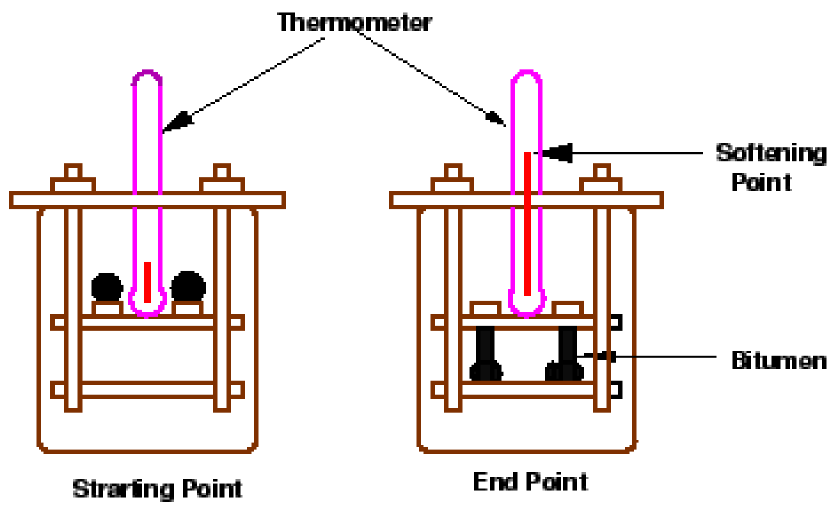

Figure 2.2: Softening point test

2.4 ASPHALT CONCRETE PROPERTIES

Asphalt concrete mixtures contain three components; air voids, mineral aggregates, and bituminous binder. The primary volumetric parameters are those relating directly to the relative volumetric proportions of these components. The various important mixture properties which show weight – volume relationship and strength are discussed here in after.

Bulk Specific Gravity Determination

The bulk specific gravity test on the freshly compacted specimens may be performed as soon as when they have cooled to room temperature. This test is conducted accordingly to ASTM D 2726, "Bulk specific gravity of compacted mixtures using saturated surface dry specimens".

In the marshal mix design procedure, the density varies with asphalt content in such way that it increases with increasing asphalt content in the mixture as the hot asphalt lubricates the particles allowing the compaction effort to force them closer together. The density reached a peak and then begins to decrease because additional asphalt cement produces thicker films around the individual aggregates, and tend to push the aggregate particles further apart subsequently resulting lower density.

The bulk density of the compacted mixture can also be altered with the proportion of mineral filler. It is expected that the bulk density increases as the amount of mineral filler increases in the mixture up to some point and then decreases. This is because an increased amount of mineral fillers will increase the amount of fines in the mix in the large amount of fine particles tend to push the larger particles apart and act as lubricating ball – bearings between these larger particles which subsequently lower the bulk density (Zemicheal, 2007).

Percent Air Void in Compacted Mixture

The air voids, Va, in a compacted paving mixture that consists of small air spaces between the coated aggregate particles expressed as percent of the bulk volume of the compacted paving mixture. To address this, HMA mix design seeks to adjust items such as asphalt content and aggregate gradation to produce design air voids.

$$Va = \left(\frac{Gmm - Gmb}{Gmm}\right)100 \tag{2.1}$$

Where:
Gmm = maximum specific gravity of the mixture, and
Gmb = bulk specific gravity of compacted mixture.

Percent Void in Mineral Aggregates in Compacted Bituminous Mixture

The voids in mineral aggregates, VMA, is the total available volume of voids between the aggregate particles in the compacted paving mixture that includes the air voids and effective asphalt content expressed as a percent of the total volume. It is calculated based on the bulk specific gravities of the combined aggregates and compacted paving mixture using the following formula.

$$VMA = 100 - \left(\frac{Gmb * Ps}{Gsb}\right) \tag{2.2}$$

Where:

Ps = aggregate as percent of total weight of mixture, and
Gsb = bulk specific gravity of aggregates.
Gmb = bulk specific gravity of compacted mixture.

The VMA has two components: the volume of voids that is filled with asphalt, and air volume remaining after compaction for thermal expansion of the asphalt cement during hot weather. It is significantly important for the performance characteristics of a mixture. For any given mixture, the VMA must be sufficiently high enough to ensure there is space for the required asphalt cement, for its durability purpose, and air space.

If the VMA is too small, there will be no space for the asphalt cements required to coat around the aggregates and this subsequently results in durability problems. On the other hand, if VMA is too large, the mixture may suffer stability problems.

Percent Voids Filled with Asphalt in Compacted Mixture

The void filled with asphalt, VFA, is a percentage of intergranular void space between the aggregate particles (VMA) that are filled with asphalt content, the amount of asphalt cements that fills the void in the mixture is termed as "effective asphalt content". It is this effective asphalt cement that provides the required asphalt film thickness around the aggregate particles, which subsequently determines the durability of the mixture.

$$VFA = \left(\frac{VMA - Va}{VMA}\right)100 \tag{2.3}$$

Marshal Stability and Flow

Marshal stability values can be determined by conducting a taste on a prepared bituminous specimen. It is the maximum load carried by a compacted specimen tasted at 60ºc applying a testing load to the specimen at a constant rate of deformation, 51mm/minute, until failure occurs. The stability value obtained is an indication of the mass viscosity of the aggregate- asphalt cement mixture. In most cases, it is affected significantly by the angle of internal friction of the aggregate and the viscosity of the asphalt cement at 60ºc. Hence, one of the easiest ways to increase the stability of an aggregate - asphalt mixture is to use a higher viscosity grade of asphalt cement. It is also possible to increase the stability of the mix by selecting a more crushed angular aggregate than rounded shape aggregates.

The flow is measured as the vertical deformation of the specimen in hundreds of inch or centimeter from start of loading up to the point where the stability begins to decrease. It is obtained at the same time at the marshal stability test is conducted. Generally high flow values indicate a plastic mix that is more prone to permanent deformation problem due to traffic loads, where as low flow values may indicates a mix with higher than normal voids and insufficient asphalt for durability and could result premature cracking due to mixture brittleness during the life of the pavement.

2.5 MARSHAL MIX DESIGN METHOD

Asphalt concrete mix design in Ethiopia is carried out based on the Marshal mix design method. The basic concepts of the Marshal mix design method were originally developed by Bruce Marshal of the Mississippi Highway Department in 1939 and then refined by the U.S. Army for use in airfield design. Army Waterways Experiment Station continued to refine the Marshal method through the 1950s with various tests on materials, traffic loading and weather variables. The Marshal method seeks to select the asphalt binder content at a desired density that satisfies minimum stability and range of flow values.

The Marshal mixture design method consists of the following basic steps:

- Aggregate selection.
- Asphalt binder selection.
- Sample preparation.
- Stability determination using the Marshal Stability and flow test.
- Density and void calculations.
- Optimum asphalt binder content selection.

The optimum asphalt binder content is selected based on combined result of marshal stability and flow, density analysis and void analysis. Based on the Asphalt Institute Method the optimum asphalt binder content is the average of asphalt contents at maximum stability, maximum density and midpoint of specified air void. Then the properties of the mix are determined at this optimum asphalt binder content. Each of the values is compared against the specification values and if all are within the specification, then the preceding optimum asphalt binder content is satisfactory. Otherwise, if any of these properties is outside the specification range the mixture should be redesigned.

2.6 RUTTING

Rutting in asphalt concrete layers develops gradually as the number of load applications increases, usually appearing as longitudinal depressions in the wheel paths accompanied by small upheavals to the sides. It is caused by a combination of densification and shear deformation (Sousa *et al*, 1994).

Densification is the further compaction of HMA pavements by traffic after construction. When compaction is poor, the channelized traffic provides a repeated kneading action in the wheel track areas and completes the consolidation. A substantial amount of rutting can occur if thick layers of asphalt are consolidated by the traffic.

The lateral plastic flow of the HMA from the wheel tracks also results in rutting. Use of excessive asphalt cement in the mix causes the loss of internal friction between aggregate particles, which results in the loads being carried by the asphalt cement rather than the aggregate structure. Plastic flow can also occur when the aggregates lack angularity and surface texture needed for adequate interparticle friction. Plastic flow can be minimized by using large size aggregate, angular and rough textured coarse and fine aggregate, stiffer binder and by providing adequate compaction at the time of construction (Roberts *et al*, 1996).

Mechanical deformation might be one of the mechanisms involved in rut development. Mechanical deformation can occur when an element under the pavement surface loses its integrity for one reason or another and is displaced under the load. A rut resulting from this type of action will generally be accompanied by substantial pattern cracking, provided the distress is allowed to progress sufficiently (Kandhal *et al*, 1998).

2.6.1 Rutting in Flexible Pavements

Rutting is a longitudinal surface depression in the wheel path accompanied, in most cases, by pavement or the underlying layer upheaval along the sides of the rut. Significant rutting can lead to major structural failure and hydroplaning, which is a safety hazard. Rutting can occur in all layers of the pavement structure and results from lateral distortion and densification. Moreover, rutting represents a continuous accumulation of incrementally small permanent deformations from each load application.

Eisemann and Hilmar studied asphalt pavement deformation phenomenon using wheel tracking device and measuring the average rut depth as well as the volume of displaced materials below the tires and in the upheaval zones adjacent to them. They concluded that:

1. In the initial stages of trafficking the increase of irreversible deformation below the tires is distinctly greater than the increase in the upheaval zones. Therefore, in the initial phase, traffic compaction or densification is the primary mechanism of rut development.
2. After the initial stage, the volume decrease below the tires is approximately equal to the volume increase in the adjacent upheaval zones. This indicates that most of the compaction under traffic is completed and further rutting is caused essentially by shear deformation, i.e., distortion without volume change. Thus, shear deformation is considered to be the primary mechanism of rutting for the greater part of the lifetime of the pavement.

2.6.2 Causes of Rutting in Flexible Pavements

Generally the causes of rutting in asphalt pavements are accumulation of permanent deformation in the asphalt surfacing layer and permanent deformation of subgrade. In the past subgrade deformation was considered to be the primary cause of rutting and many pavement design methods applied a limiting criteria on vertical strain at the subgrade level. However recent research indicates that most of the rutting occurs in the upper part of the asphalt surfacing layer. These causes of rutting can act in combination, i.e., the rutting could be the sum of permanent deformation in all layers (Garba, 2002).

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2.6.2.1 Rutting Caused by Weak Asphalt Mixture

Rutting resulting from accumulation of permanent deformation in the asphalt layer is now considered to be the principal component of flexible pavement rutting. This is because of the increase in truck tire pressures and axle loads, which puts asphalt mixtures nearest the pavement surface under increasingly high stresses.

Brown and Cross reported on an extensive national study of rutting in hot mix asphalt pavements in United States. The study was initiated in 1987 to evaluate pavements from all areas of the United States encompassing various climatic regions, containing aggregates of differing origins and angularity, encompassing different specifying agencies and construction practices and a large sample size to make the study results national in scope. The study involved collection of pavement core samples for material characterization, measurement of rut depth and layer thicknesses, and investigation to determine the location of rutting. The conclusion from this study regarding the location of rutting was that the majority of rutting was occurring in the top 3 to 4 inches (75 to 100 mm) of the asphalt concrete layers. They found that the rutting in the subgrade was generally very small.

It is thus abundantly clear that rutting caused by accumulation of permanent deformation in asphalt layers is the primary cause of flexible pavement deterioration. To reduce this form of deterioration it is necessary to pay more attention to the selection of materials and mix design. To be able to design mixtures that have adequate resistance to rutting, the effect of mixtures' volumetric composition and properties of the component materials on their permanent deformation response must be clearly understood. Further, there should be a simple measure of resistance of mixtures to rutting that can be used at mixture design stage to enable evaluation and selection of rut resistant mixtures. This issue is the main areas focus of this thesis work.

Rutting in asphalt layers is caused by an asphalt mixture that is too low in shear strength to resist the repeated heavy loads to which it is subjected. Asphalt pavement rutting from weak asphalt mixtures is a high temperature phenomenon, i.e., it most often occurs during the summer when high pavement temperatures are evident. Figure 2 illustrates rutting caused by weak asphalt mixture.

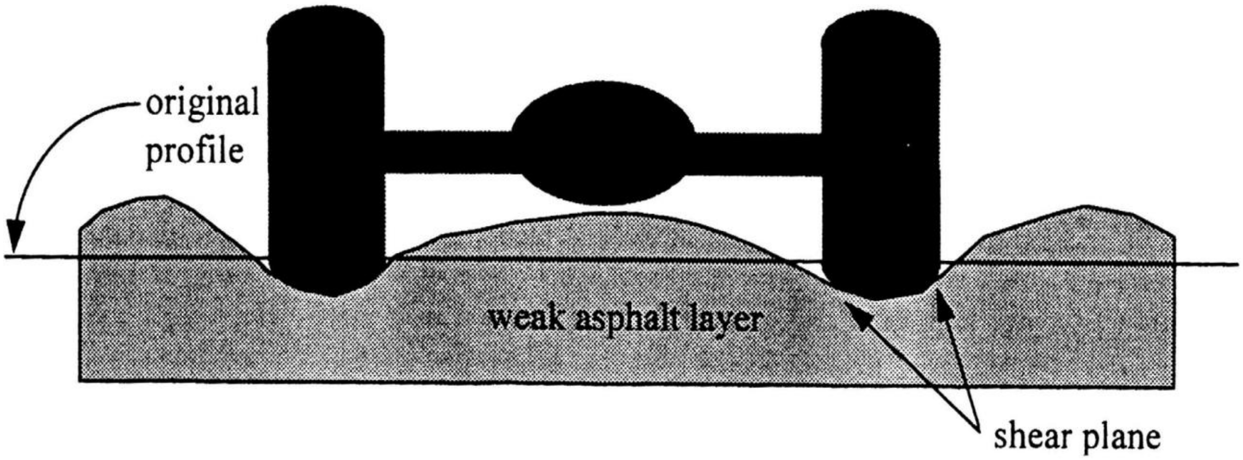

Figure 2.3 Rutting caused by weak asphalt layer

2.6.2.2 Rutting Caused by Weak Subgrade

Rutting can be caused by too much repeated load applied to subgrade, sub base or base below the asphalt layer. In many cases this is due to insufficient depth of cover on the subgrade resulting from too thin an asphalt section to reduce the stress from applied loads to tolerable level. Thus this type of rutting is considered to be more of a structural problem than a materials problem and is often referred to as structural rutting. Intrusion of moisture can also be the cause for weakening of the subgrade. In this type of rutting, the accumulated permanent deformation occurs in the subgrade. Figure 2.4 illustrates rutting from weak subgrade.

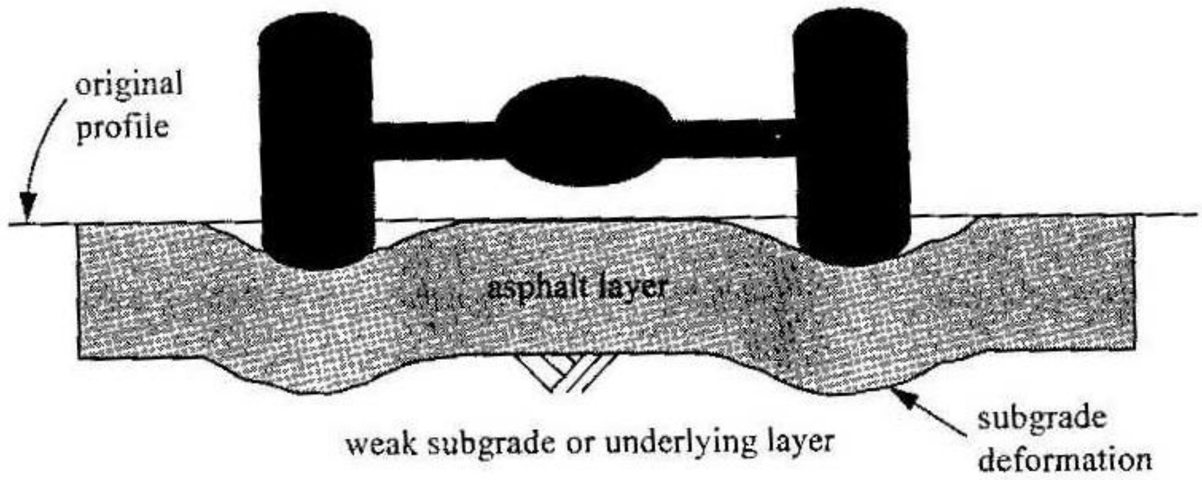

Figure 2.4 Rutting from weak subgrade

2.6.3 Rutting Consideration in Pavement Design

Mainly empirical methods were used to design pavements. These methods do not consider pavement distress explicitly. In recent years, the more rational mechanistic – empirical methods have been developed and are being implemented in some countries. Generally two procedures have been used in the mechanistic - empirical methods to limit rutting: one to limit the vertical compressive strain on top of the subgrade and the other to limit the total accumulated permanent deformation on the pavement surface based on the permanent deformation properties of each individual layer. Given that with increased tire pressures most of rutting occurs in the asphalt surfacing layer rather than the subgrade, the first approach appears inappropriate for consideration of rutting in pavement structural design.

In the second approach, the permanent deformation properties of each individual layer are taken into account. This requires testing and characterization of the materials used in the pavement structure. It also requires the calculation of stresses at selected points in each layer. The permanent deformation of each layer is then calculated and summed up to find the total permanent deformation. This approach is rational and it allows the explicit consideration of permanent deformation properties of materials in each layer.

2.6.4 Rutting Consideration in Mixture Design

The purpose of mix design is to determine the proportions of aggregate and binder that would produce a mix, which is economical and has the following desirable properties:

- Sufficient binder to ensure durability
- Sufficient voids in mineral aggregate, so as to minimize post construction compaction without loss of stability and without causing bleeding, and to minimize harmful effects of air and water.
- Sufficient workability to permit laying of the mix without risk of segregation, and
- Sufficient performance characteristics over the service life of the pavement

Among the different mix design method the most widely used in Ethiopia is the Marshal mix design method. The marshal method seeks to select the asphalt binder content at a desired density that satisfies minimum stability and range of flow values. The variables optimized in Marshal mix design methods are not direct measure of performance. For instance the Marshal stability is a surrogate measure of mixture's shear strength. The Marshal flow is specified to limit permanent deformation. But the Marshal method has several shortcomings including:

1. The impact hammer used to prepare specimens in this method does not simulate the compaction that occurs in pavements, and
2. It is not suited to the present day traffic conditions as evidenced by the steady increase in rutting problems in recent years with mixes prepared using this method.

Marshal mix design method do not properly evaluate the rutting resistance of asphalt concrete mixtures. It appears that the resistance to rutting of different asphalt concrete mix produced by different binder grades and modifiers has to be evaluated prior to construction of the asphalt concrete surface. This issue forms the area of focus in this study.

2.6.5 Influence of Asphalt Cement Properties on Rutting

Asphalt Cement (bitumen) is a viscoelastic material with suitable mechanical/rheological properties for traditional paving applications because of their good adhesion properties to aggregates. Bitumen is a material characterized by a time of loading and temperature dependence of the mechanical response to loading (Hafeez, 2009). However, the increase in traffic load requires improving the mechanical properties of conventional asphalt mixtures. The chemical composition of the AC has a significant effect on its viscoelastic properties and hence on its performance as road paving material in asphalts.

The behavior of asphalt at high temperature conditions for short time spans is equivalent to its performance at low temperature conditions for longer time durations. This concept is called temperature shift or in other words the superposition theory of asphalt binder, which has been explained by Asphalt Institute (2003) as in Figure 2.5.

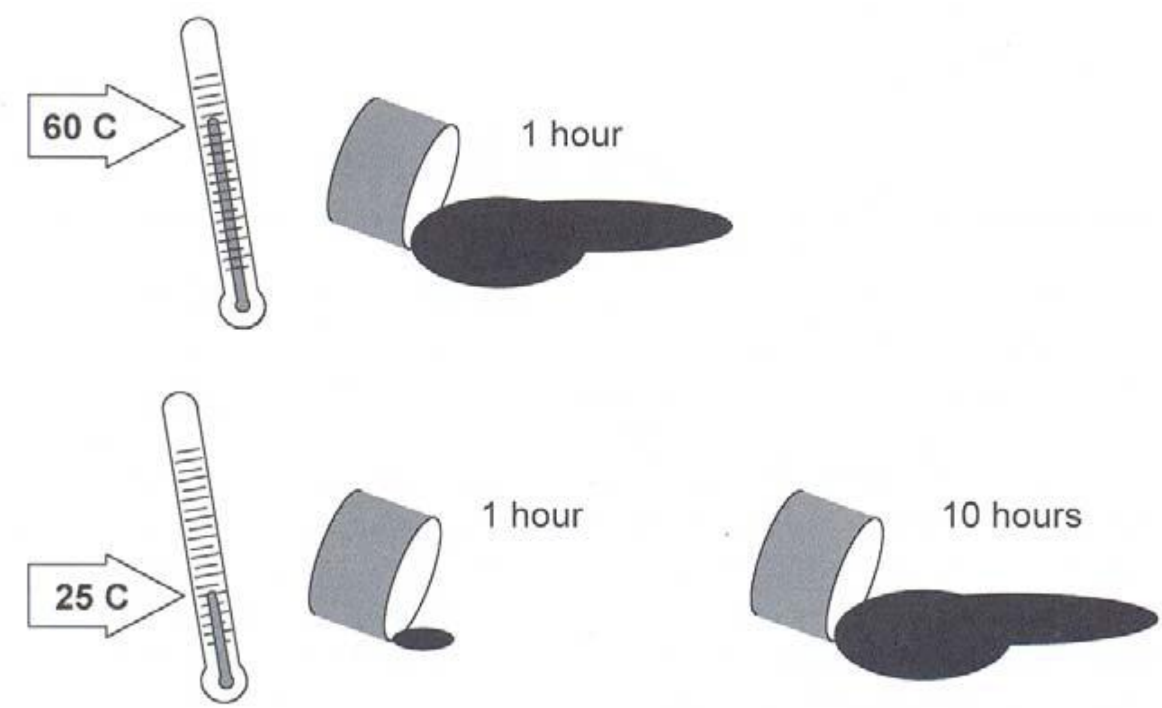

Figure 2.5: Temperature Shift Behavior of Asphalt Binder (Asphalt Institute, 2003)

In hot climatic conditions or under slow moving trucks, asphalt behaves like a viscous liquid and only aggregates are the contributing element of hot mix asphalt that bear the traffic loads. Whereas in cold climatic conditions or under fast moving trucks (rapidly applied loads), asphalt behaves like an elastic solid and deforms when loaded, but returns to its original shape when unloaded. If it is stressed beyond its strength, it may rupture.

At intermediate temperature conditions, asphalt binder exhibits the characteristics of both viscous liquids and elastic solids. Due to this property of asphalt, it is considered to be an excellent adhesive material for use in paving. When heated, asphalt acts as a lubricant, allowing the aggregate to be mixed, coated, and tightly-compacted to form a smooth and dense surface.

After cooling, it acts as a glue to hold the aggregate together in a solid matrix. In its finished state, the behavior of the asphalt is termed as visco-elastic i.e., it has both elastic and viscous characteristics, which depends on the temperature and rate of loading. The elastic response or recoverable part, viscoelastic response and plastic response or non-recoverable which appears in the form of permanent deformation have been illustrated in Figure 2.6 (Hafeez, 2009).

Rutting at high temperatures, crack initiation and propagation in the low temperature region and other forms of pavement defects are not only due to traffic loads but also due to the capability of the asphalt concrete to sustain temperature changes. Increased traffic factors such as heavier loads, higher traffic volume, and higher tire pressure demand higher performance pavements. High performance pavement requires AC that is less susceptible to high temperature rutting or low temperature cracking, and has excellent bonding to stone aggregates according to Ait-kadi et al, 1996 (Hafeez, 2009).

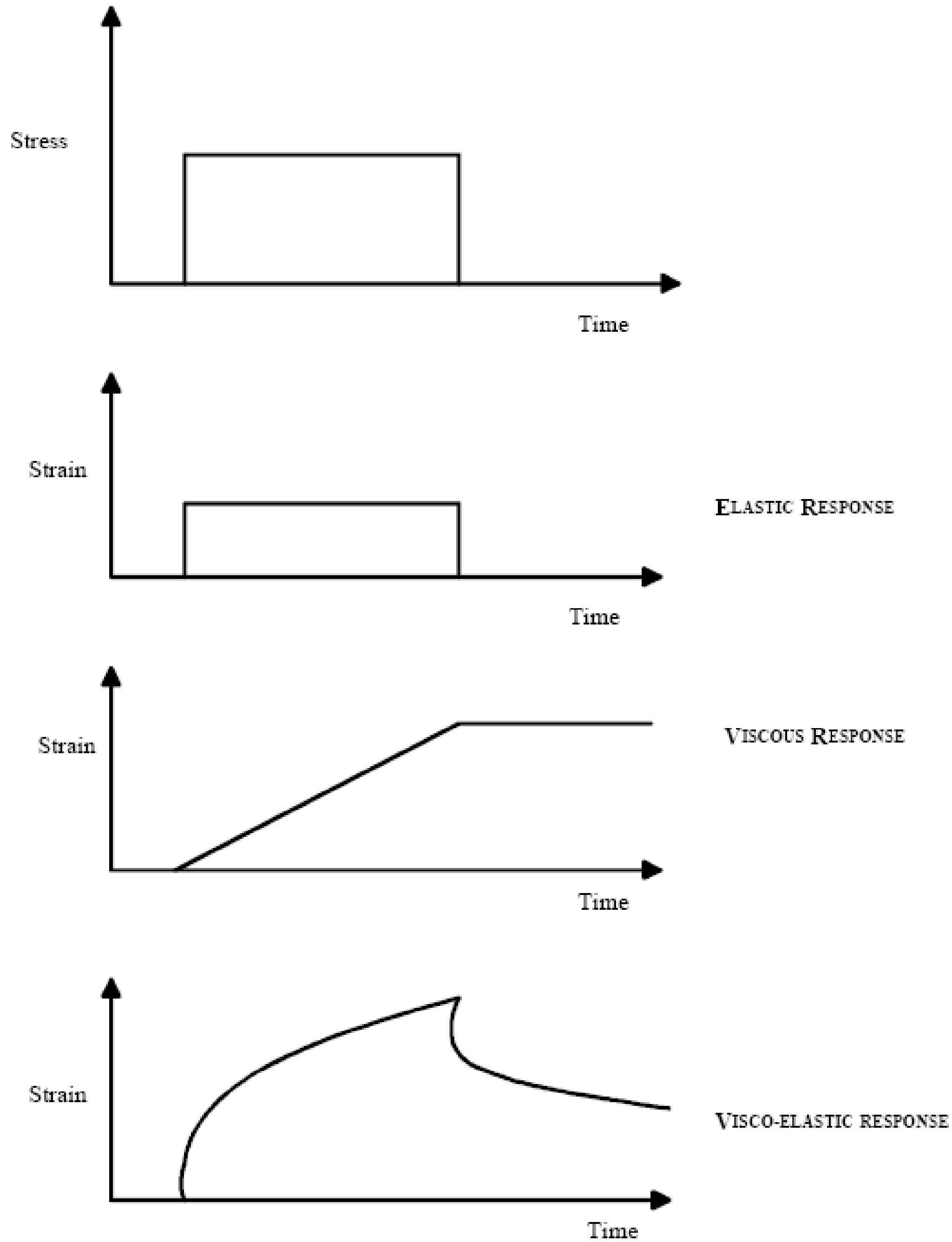

Figure 2.6: Stress Strain Behavior of Bituminous Material (Hafeez, 2009).

William, et al., (1967) studied the influence of rheological properties of asphalt on rate of deformation and strength of asphalt concrete and reported a direct relation between them. The asphalt viscosity directly affects the strength of asphalt concrete in compression (rutting) for the practical range of temperatures. The log of pavement resistance and of cohesion varies directly with the log of asphalt viscosity. Modulus of elasticity in compression was influenced by the type of asphalt, temperature and amount of lateral confinement.

The increase in deformation is related to the decrease in binder viscosity at high temperatures, thereby leading to a lower interlock between the aggregates. The contribution of the aggregate skeleton towards the behavior of the mixture becomes more significant at higher temperatures (Hafeez, 2009).

Mahboub and Little (1988) found that mixtures containing less viscous asphalts are less stiff and are more prone to rutting. They also recommended more viscous asphalt cements in thicker pavements and hotter climates on the basis of similar observations.

Monismith and Tayebali (1988) examined the relative behavior of mixtures with and without modifiers. They found that mixtures containing modified asphalt cement showed better resistance to rutting at high temperatures than the mixture containing the neat asphalt cement. They also reported that resistance to rutting may be improved by the use of modifiers (polymers, micro fillers, etc.) which make asphalt binder more viscous at higher temperatures without any adverse effect at low temperature.

As an alternative to larger top size and coarser mixes, polymer-modified asphalt cement and other modified asphalt cement products are also being investigated by many agencies to increase the resistance to permanent deformation (Prowell, 1999)

2.7 MODIFIED BINDERS

Asphalt binders have a limited capacity to perform when under wide range of loads and weather conditions which occur over the life of a pavement (Chen et al, 2002). Therefore, binders are modified to improve their performance. One of the prime roles of a bitumen modifier is to increase the resistance of asphalt to permanent deformation at high road temperatures without adversely affecting the properties of the bitumen at other temperature. This is achieved by stiffening the bitumen so that the total visco-elastic response of the asphalt is reduced or by increasing the elastic component of the bitumen thereby reducing the viscous component. The use of modified bitumens offers a solution to reducing the frequency of maintenance required at particular locations and provides a much longer service life for maintenance treatments at difficult sites.

Based on their functions and behaviors, polymers can be divided into three types according to Khattak, 1998 (Zaniewski et al, 2003):

- Dispersed thermoplastics such as Polyethylene,
- Network thermoplastics such as Styrene-Butadiene-Styrene, SBS, and
- Reacting polymers such as Elvaloy AM.

Chen et al, (2002) studied the properties of SBS polymer-modified asphalt. The engineering properties of the asphalt modified by SBS showed an increase in stiffness as a function of SBS copolymer. Because of the colloidal nature of asphalt cements, their mechanical properties were highly enhanced after SBS modification due to the presence of the dispersed phase, and swelling of the polymer. The minimum percentage of polymer to ensure the formation of its continuous phase depends to a greater extent on the base asphalt and the polymer itself.

The SBS triblock copolymer is one of the most promising polymers for asphalt modification (Khattak et al, 1998). SBS is an inhomogeneous material; the engineering properties are strongly influenced by the morphology of the composite. The microstructure of polymer-modified binders, PMA is related to the characteristics of each constituent that forms the material.

According to Khattak et al, (1998) who did a study under laboratory conditions on the structural and engineering properties of PMA, modified with SBS polymer system indicated considerable increase in indirect tensile strength and fracture toughness of asphalt mixtures at 25ºC and 60ºC. This implies increased resistance to fatigue cracking and rutting. The higher number of load cycles to develop plastic deformations and the almost constant resilient modulus indicate that the SBS polymer system cause a decrease in the energy stored in the sample due to plastic deformation. The fatigue life of PMA mixtures is considerably higher than for straight and processed asphalt mixtures. The increase in fatigue life is due to increases in tensile strength and plastic properties of the mixes.

Elvaloy is a random terpolymer comprising ethylene, normal butylacrylate and glycidyl methacrylate (GMA). The molecular weight and co monomer levels can be varied during polymer manufacture. Added in small quantities to asphalt, Elvaloy terpolymer creates a permanently modified binder with improved elastomeric properties. Unlike most other plastomers and elastomers that are simply mixed into asphalt, Elvaloy has an active ingredient that chemically reacts with asphalt. The result is not a mixture of asphalt and modifier, but rather a stable, elastically improved, more resilient binder that can be stored and shipped to hot mix plants to help meet higher-performance specifications. Elvaloy copolymers chemically react with asphalt to form a polymer-linked-asphalt system with improved performance properties.

Hot mix asphalts made with Elvaloy are easy to spread and compact, and provide outstanding resistant to rutting, cold cracking and fatigue. Roads made with Elvaloy have been in service since 1991, and are showing excellent long-term durability (Zaniewski et al, 2003).

NES is one of the modifiers produced in china for those areas with high temperature and/or high moisture. Among the different types NES, HSA, high modulus modifier is produced for highly loaded highways and hot areas. The elements used to produce HSA (NES-1) are natural rock asphalt, performance reinforcing agent, stability agent and surfactant. After mixing HSA into bitumen there is a Chemical reaction which

- improve the vander waals' force among bitumen molecules

- brings a large number of polar bonds which transforms molecular groups such as wax, naphthaline, etc.

- producing colloid cross-linking thus to form net structure

The expected improvements after using HSA modifiers are absorbability between bitumen and aggregate, high temperature resistance and adhesion of base bitumen, and water damage resistance of bitumen aggregate. (Zhao-feng et al, 2008)

Mechanism of NES Bitumen Modification

When the NES bitumen modifier is added into the base bitumen, the joint effect of temperature and small molecules of solvent makes the molecules of natural bitumen micelles rupture, and many active sites on the exposed fracture of the micelle immediately filled and saturated by small molecules and formed a new composition. Gradually, the bitumen will be formed in a new way with natural molecular micelle centered, small molecules filled and surrounded. The modifying degree depends on the molecular weight and the amount of nitrogen, oxygen and sulfur.

According to Zhao-feng et al, (2008) who did a study on Performance Analysis on Modified Bitumen by NES Natural Rock Asphalt indicated when the mixing of NES rock asphalt modifier, the penetration of modified asphalt will decrease, softening point will rise and ductility will significantly reduce. It indicates that the asphalt modified by NES rock asphalt becomes stiff, resisting deformation and enhancing high-temperature performance, and temperature sensitivity significantly reduces at the same time. At the same temperature, the complex shear modulus, creep stiffness modulus and the fatigue resistance factor of modified asphalt gradually increases with the rock asphalt addition.

2.8 EQUIPMENT TO EVALUATE RUTTING

Permanent deformation or rutting can be evaluated by using equipments like Asphalt Pavement Analyzer, Figure 2.7. Asphalt Pavement Analyzer, APA, is multifunctional loaded wheel tester used for evaluating permanent deformation (rutting), fatigue cracking, and moisture susceptibility of both hot and cold asphalt mixes. The priority was to use APA to carry out the respective laboratory works of the study.

However, APA cannot be used since it is not available in Ethiopia. Instead of APA, a Chinese rut testing device called ’’Quan Zi Dong Che Zhe Shi Yan Yi (QZDCZSYY)’’, figure 2.8, which can also perform the tests required for this study was used. The device has been used in china for more than 20 years. This device features controllable temperature, wheel load and contact pressure that are representative of actual field conditions. A typical testing time for a complete evaluation is 60 minutes (2520 cycles). Rutting susceptibility of the mixes is evaluated by placing samples under repetitive loads. The standard method followed to determine rutting susceptibility using QZDCZSYY is given as JTJ 052-2000 T0719-1993 in the Chinese Standard, Standard Test Method of bitumen and Bituminous Mixture.

The QZDCZSYY rut testing device has and use the following components to perform the test based on Standard Test Method of bitumen and Bituminous Mixture of the Chinese Standard.

Loading device: It shall make the contacting pressure between test wheel and test piece be 0.7MPa±0.05MPa, and the applied total load is about 78kg, which may be adjusted as it is required.

Specimen mould: Made up of steel plate, it is composed of bottom plate and side plate. Its internal length is 300mm, width is 300mm, and thickness is 50mm.

Deformation measuring devices: Devices that automatically detect the rut deformation and record curve, which generally are electrical logging dial indicator or non-contacting displacement meter.

Temperature-detecting devices: The temperature transducer and thermometer that automatically detect and record the temperature at test piece surface or in thermostatic chamber, which are accurate to 0.5ºc.

Thermostatic chamber: Rut testing machine must be wholly settled in thermostatic chamber; the thermostatic chamber is equipped with heater, airstream circulating device and automatic temperature controlling equipments that are able to keep the temperature in thermostatic chamber at the required temperature. It is used for the thermal insulation of test piece and with test carried on in it. Its temperature shall be able to be self-recorded automatically.

Figure 2.7 Asphalt Pavement Analyzer

Figure 2.8 Quan Zi Dong Che Zhe Shi Yan Yi –The Chinese Rut Testing Device

Comparison of QZDCZSYY against APA

- Both APA and QZDCZSYY features, controllable wheel load, contact pressure and pavement temperature that are representative of actual field conditions.
- APA can evaluate rutting, fatigue cracking and moisture susceptibility of asphalt concrete mixes but QZDCZSYY can only evaluate rutting.
- Rutting susceptibility of the mixes is evaluated for both beam and cylindrical samples for APA but only beam samples are evaluated for QZDCZSYY.
- APA can test three to six samples together although QZDCZSYY can only test one sample at a time.
- APA can test in dry or submerged conditions however QZDCZSYY can only test in dry condition.

2.9 CONCLUSIONS

The different grading system of binders and tests for binder properties were described in this chapter. It is also stated in the literature review that it is very much pertinent to consider the properties of bituminous binders and the bitumen content in a mix for enhancing the performance characteristics of bituminous mixes. The literature review demonstrates seriousness of the rutting problem. Furthermore, rutting in the asphalt pavements is a complex phenomenon, dependent on several factors. Several authors have demonstrated that modified asphalt binders may significantly assist in controlling rutting.

CHAPTER 3 RESEARCH METHODOLOGY

3.1 INTRODUCTION

This research evaluates the effect of binders and modifiers with respect to rutting performance. Starting with mix designs from China Communication and Construction Corporation different grade binders were substituted and tested to evaluate the effect of binder type on rutting potential. The mix design was prepared with 40/50 binder. The binders evaluated were 60/70 and 40/50 with and without modifier. These are among the binders used in Ethiopia. The mix design was based on Marshal method. Samples were made using the acquired mix design and were later tested in the Quan Zi Dong Che Zhe Shi Yan Yi (QZDCZSYY). The 13mm mix or the wearing course was used for the evaluation of the binders. The following sections of this chapter explain the laboratory testing program.

3.2 MATERIALS

The aggregate used in this research work is provided by China Communications Construction Co. Ltd, CCCC. The 12.5 mm mix consisted of aggregates crushed to size less than 12.5mm. During construction, four stockpiles of materials were used to create the mix. These are 0-3mm, 3-5mm, 5-10mm and 10-12.5mm.

The asphalt binders used were supplied by China Communications Construction Co. Ltd, The binders were 60/70 and 40/50. The modifier used was supplied by SDHS Qingchuan Road Materials Development Co. Ltd in association with Rosnar Holding Co. Ltd. The modifier used was NES which is shown on Fig. 3.1. The proportion of the modifier used as supplied by the vender was 0.4-0.6% of the total weight of the asphalt concrete mix. The selection of the proportion is based on the level of improvement required and economy. For this study 0.4% was taken in order to show the improvements at the economical choice. The mixing and compacting temperatures for each binder grade are given in Table 3.1.

Figure 3.1 NES- The Modifier

Table 3.1 Mixing and compaction temperatures

Grade of bitumen	Mixing Temperature (min.-max.)	Compaction Temperature (min.-max.)
60 – 70	150 – 185 ºc	90-170 ºc
40 – 50	160 – 185 ºc	100-170 ºc

The mixing and compaction temperature taken for this study was 185ºc and 170ºc respectively. The bitumen and aggregate were heated to temperature of 170ºc and 180ºc respectively before mixing.

3.3 BINDER PREPARATION

The binders supplied to the lab were first sampled following the specifications given in AASHTO T 40. Samples were heated in the oven until it was suitable for pouring. It was then quartered into different containers and stored for further testing.

3.4 TESTS ON BINDERS

The penetration test and softening point test were conducted to get the properties, especially the consistency of each binder grade. As the penetration grading system is based the penetration values, the penetration value for 40/50 binder type should be between 40 and 50 and for 60/70 binder type should be between 60 and 70. The softening point and penetration values of 60/70 and 40/50 binder grades are tabulated in table 3.2.

Table 3.2 Test result for penetration and softening point tests

Grade of bitumen	Penetration (0.1mm)	Softening point (ºC)
40/50	46	53
60/70	65	47

3.5 AGGREGATE PREPARATION

The aggregates were processed by sieving, washing and oven drying. Dried aggregates were sieved with a nest of sieves, consisting of 50 mm, 37.5 mm, 25 mm, 19 mm, 12.5 mm, 4.75 mm, 2.36 mm, 1.18 mm, 600 μm, 300 μm and 75 μm. The material retained in each sieve was washed and placed in storage bins. The pan material from the dry sieving was placed in the storage bins. Dust material removed when washing was discarded. Sieving is done only to get material to prepare asphalt concrete samples, and not for gradation analysis.

3.5.1 Specific Gravity of Aggregates

Two samples of each aggregate were tested to determine the specific gravity and absorption, AASHTO T 85 for coarse aggregate, and AASHTO T 84 for fine aggregate specifies the procedures for determining specific gravities. Sample was split following the specifications in AASHTO T 248. Two samples were taken from each bag and the tests were done to determine the specific gravity and the absorption percentage. The average specific gravity and absorption values determined in the laboratory are given in Table 3.3.

Table 3.3 Specific gravities of aggregates for 12.5mm mix

Kinds of Material	10-16mm	5-10mm	3-5mm	0-3	Filler
Apparent specific gravity	2.978	2.992	2.806	2.999	2.866
Bulk specific gravity	2.832	2.839	2.591	2.772	
Water absorption (%)	1.72	1.80	2.74	2.96	
Material Proportion (%)	29.0	30.0	3.0	37.0	1.0

3.5.2 Aggregate Blend

The aggregate blend was designed by China Communications Construction Co. Ltd., CCCC, for the construction of Addis Ababa – Adama Toll Motorway Project. The same blend was used for this study to make specimens for rut depth measurement using the different binders. The design aggregate blends are summarized in Table 3.4.

Table 3.4 Gradation of 13mm Mix

Sieve Size(mm)	% Passing(Result of Sieve Analysis)			
	Design Proportion(%)	Target Proportion(%)	Upper limit(%)	Lower limit(%)
19	100	100	100	100
16	100	100	100	100
13.2	96.6	96.3	100	90
9.5	76.2	78	85	68
4.75	47.2	48.4	68	38
2.36	32.6	30.4	50	24
1.18	21.8	22.4	38	15
0.6	16.2	16.6	28	10
0.3	10.7	11.3	20	7
0.15	8.2	9.1	15	5
0.075	6.9	7.9	8	4

The design gradation shows some difference from the target gradation. However, all results fall within the upper and lower control requirements of the specification.

3.6 ASPHALT CONCRETE MIX DESIGN

The asphalt concrete mix design was based on Marshal method. The asphalt concrete mix was designed by China Communications Construction Co. Ltd., CCCC, for the construction of Addis Ababa – Adama Toll Motorway Project. The same mix design was reviewed and used for this study. The optimum asphalt binder content was 4.58% of the total weight of the mix. The design gradation was given in the table 3.4.

3.7 THEORETICAL MAXIMUM SPECIFIC GRAVITY

The materials were proportioned according to the mix design formula to make theoretical maximum specific gravity, G_{mm}, (AASHTO T 209) samples. G_{mm} was determined for two samples for each mix and binder to check variability. The values of G_{mm} for these samples were within the precision limits as specified in AASHTO T 209. Table 3.5 presents the average G_{mm} for the four mixes using the different binders. Detailed G_{mm} data for the samples are presented in Appendix A.

Table 3.5 Average G_{mm} of HMA for the Four Mixes

Asphalt Grade	G_{mm}
60/70	2.641
60/70+Modifier	2.643
40/50	2.644
40/50+Modifier	2.644

3.8 BULK SPECIFIC GRAVITY

Materials were proportioned according to mix design to make samples for checking bulk specific gravity, G_{mb}. AASHTO T 166 standard method was followed. G_{mb} was checked for the samples prepared in the laboratory. The samples were compacted in the mould and compacted 75 times both sides based on Marshal method. Five samples were produced based on the mix design for each type of mix. Table 3.6 presents average G_{mb} based on the laboratory results presented in Appendix A.

Table 3.6 Average G_{mb} of HMA for the Four Mixes

Asphalt Grade	G_{mb}
60/70	2.508
60/70+Modifier	2.525
40/50	2.526
40/50+Modifier	2.529

3.9 PREPARING SPECIMEN FOR QZDCZSYY TEST

The materials for making the specimens were proportioned according to the mix design. These proportions depend on the weight of the sample, which depends on the mix type, which in turn depends on the value of the G_{mb}. Table 3.7 demonstrates that the G_{mb} changes for each mix and also for different binder. The sample weight was estimated taking into consideration the volume of the specimen after compaction i.e.the volume of the mould. The dimension of the mould is 30cm*30cm*5cm. The following equations were used to estimate the weight of the samples.

$$Mm = \text{Mould Volume} * Gmb * 1.03 \tag{3.1}$$

Where,

Gmb- bulk specific gravity

Mm- weight of the sample

The value was multiplied by 1.03 to consider lost HMA while placing in the mould. Based on the above formula the weight of the samples of each mix type was calculated and tabulated in table 3.7.

Table 3.7 Weight of the Different HMA for the Specified Mould

Binder Grade	Wt(gm)
60/70	11624
60/70+Modifier	11703
40/50	11708
40/50+Modifier	11722

To produce the HMA, the aggregate samples were heated to the mixing temperatures corresponding to each binder. The asphalt binder was heated to the desired temperature and weighed into the heated aggregate. Mixing is done in a large mechanical mixer to thoroughly mix the asphalt binder and aggregates. The aggregate and the binder were mixed in a heated mixer. Finally, filler was weighed in to the heated mix and was mixed until a homogeneous mixture is achieved.

When modifier (NES) is used, there are two methods of mixing the modifier with the asphalt aggregate. These are Wet mixing method and Dry mixing method.

Wet mixing method - the mixing processes are

- Heat base bitumen to 175°C.
- Slowly add the modifier (NES) into the base bitumen.
- Agitate for 0.5-1hour and get end product: NES modified bitumen.
- Store final product into tank with agitator and transfer them directly to mixing station to mix with aggregates.

Dry mixing method- The modifier was mixed with the mixed aggregate and binder before the filler was added.

Even though wet mixing method is expected to provide more improved AC mixture, dry mixing method was used for this study. It is because the equipment used in wet mixing method was not available.

After mixing is complete, the loose mix is placed in an oven at compaction temperature to induce short term aging. Short term oven aging simulates the induced aging during production and placement of HMAC. Short term aging period of 2 hours was taken. During the short term aging period, the specimens are stirred to ensure uniform aging throughout the mix. The sample was transferred into moulds and placed for compaction. The moulds are heated in the oven at the specified compaction temperature to ensure that the mix temperature is not reduced. The amount of loose mix required for specimen preparation is calculated based on equation 3.1. The estimated weight is placed into the mold and the specimen is then compacted. A round shaped or roller like Chinese equipment called Che Zhe Cheng Xing Yi (Compacting Machine) as shown in the figure 3.2 with pressure of 3MPa was used to compact the specimen which simulate the field compaction. The sample is compacted using this equipment 18 times in one direction and another 18 times in the other direction at the compaction temperature to reach the required compactness i.e. 100% of the Marshal standard compactness. All the samples were compacted equally. The compacted specimens are cooled to room temperature for a period of at least 24 hours before QZDCZSYY test. As for modified bituminous mixtures, they should be laid at normal temperature for at least 48 hours before it is used in rutting test, the modified bituminous mix takes longer time to solidify sufficiently.

Figure 3.2: Che Zhe Cheng Xing Yi (Compacting Machine)

3.10 TEST PROCEDURE FOR QZDCZSYY TEST

The Quan Zi Dong Che Zhe Shi Yan Yi (QZDCZSYY) test was designed for slab specimen having a dimension of 300 mm length 300mm width and 50mm height. Specimens were made using the 13 mm mix which is mixed in the laboratory. Four specimens were made for each mix type for the test.

The QZDCZSYY tests only one specimen at a time with a reciprocating solid rubber wheel. The wheel has an outside diameter of 200 mm and a width of 50 mm. The average contact stress given by the manufacturer is 0.7 MPa and the applied total load is about 78kg, which may be adjusted as it is required. Given that the contact area increases with rut depth, contact stress is variable. According to the manufacturer, a contact stress of 0.7 MPa approximates the stress produced by one rear tire of a double-axle truck. As for the test wheel, its traveled distance is 230mm±10mm. The average speed of the wheel is 42 ± 1 wheel passes per minute (21 reciprocations per minute).

Information regarding the specimen and test temperature is entered into the computer. The test temperature is the maximum pavement temperature of the proposed road or area of use, 60ºc was used for this study. The specimens are placed in the oven at the specified test temperature for 5 hours. When the test temperature is reached in the QZDCZSYY, the specimen from the oven was moved to the QZDCZSYY as quickly as possible to maintain the test temperature. Figure 3.3 shows specimen inside the QZDCZSYY while testing is under progress. Once the test starts, the wheel starts moving at the specified average speed. The test is automatically stopped after one hour. The rut depth for the specimen is recorded at 45 minute and 60 minute. Also in the test, the recording instrument automatically records the deformation curve (deformation Vs time) and the temperature of test piece.

The deformation is measured at the center of the specimen since it was identified that measuring the rut depth at the center of the specimen provides a more repeatable rut depth measurement. Maximum rut can occur at any point on the specimen but is usually seen in the central area of the specimen because there is less confinement in this area as opposed to the rest of the specimen.

The test result also includes dynamic stability of the specimen which is based on the rut depths at two different time, 45 minute and 60 minute are taken for this study. According to the manufacturer, the dynamic stability is the number of repetition of the wheel to make a rut depth of 1mm. It is calculated by the following equation.

$$\text{Dynamic Stability (times/mm)} = \frac{((T_2 - T_1) * \text{Wheel average speed})}{(\text{Rut depth at } T_2 - \text{Rut depth at } T_1)}$$

Where,

Wheel average speed=42times/min

T_1 = 45minute

T_2 = 60minute are taken for this study.

The higher the dynamic stability the less susceptible the mix to rutting. According to Chinese specification, the dynamic stability of the mix should be greater than 3000times/mm. The dynamic stability and rut depths data for all specimens tested are presented in Appendix B.

Figure 3.3 Specimen in QZDCZSYY While Testing

CHAPTER 4 RESULTS AND DATA ANALYSIS

Using the data as reported in Chapter 3 the results of the test were analyzed. First the volumetric properties of the mix were evaluated in order to check the quality of the mix. Then statistical analyses of the rut depth or QZDCZSYY test results were performed.

4.1 MARSHAL PROPERTIES

Samples with the mix design were prepared to check the volumetric properties, stability and flow of the mixes with different binder grades. The results are summarized in Table 4.1. The volumetric properties, stability and flow for the specimens produced from all four types of mixes meet the criteria. However, the volumetric properties, stability and flow of these mixes are different. For the mix produced from 60/70 binder grade which do not have modifier, the air void and flow meet the criteria marginally i.e. the air void is 5% and flow is 3.5mm. AC mix produced from 40/50 binder grade has higher stability than the Asphalt concrete mix produced from 60/70 binder grade. This is because when the viscosity of the asphalt cement increases the marshal stability increases. Comparatively, the Asphalt Concrete mix produced from 40/50 binder grade has higher bulk specific gravity than the Asphalt Concrete mix produced from 60/70 binder grade. Effects of binder grades and modifiers on marshal properties are described herein after.

Table 4.1 Volumetric Properties, Stability and Flow Values of Mixes

Properties	Criteria	Without modifier	With modifier (NES)
Mix with 60/70 binder grade			
% Binder	-	4.58	4.58
Stability(KN)	min 8KN	15.6	16.4
Flow(mm)	1-3.5mm	3.5	2.67
Va %	3-5	5.0	4.47
VMA %	min 14.0	14.6	14.1
VFA %	65-75	65.7	68.2
G_{mm}	-	2.641	2.643
G_{mb}	-	2.508	2.525
Mix with 40/50 binder grade			
% Binder	-	4.58	4.58
Stability(KN)	min 8KN	16.1	17.68
Flow(mm)	1-3.5mm	3.47	2.89
Va %	3-5	4.45	4.33
VMA %	min 14.0	14	14.0
VFA %	65-75	68.3	68.8
G_{mm}	-	2.644	2.644
G_{mb}	-	2.526	2.529

Effect on Bulk Density

As shown in figure 4.1 the AC mix produced from 40/50 binder grade has higher bulk density than the Asphalt concrete mix produced from 60/70 binder grade. Furthermore, the Asphalt Concrete mix with modifier produced from 60/70 and 40/50 binder grades have higher bulk density than the Asphalt Concrete mix without modifier produced from 60/70 and 40/50 binder grades respectively.

Even though the bulk density of asphalt concrete mix increased with the addition of modifier, the increase in bulk density was not significant when modifier was added in the asphalt concrete mix with 40/50 binder grade.

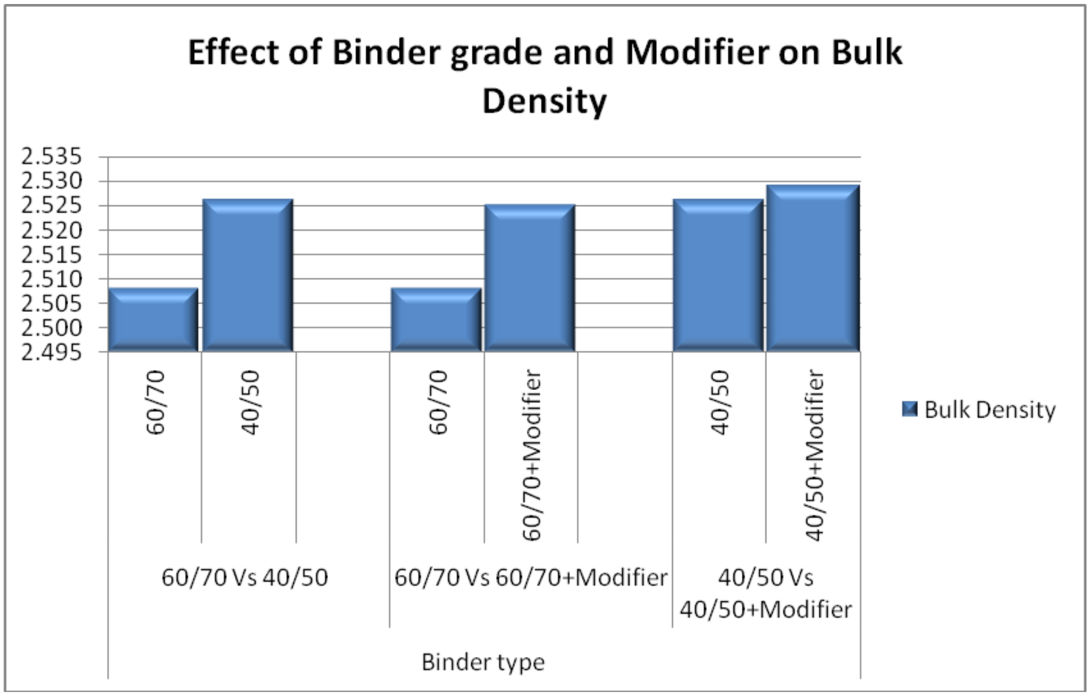

Figure 4.1 Effect of Binder grade and Modifier on Bulk Density

Effect on Marshal Flow

As shown in figure 4.2 the AC mix produced from 40/50 binder grade has lower flow than the Asphalt concrete mix produced from 60/70 binder grade. This is because when the viscosity of the asphalt cement increases the marshal flow decreases. Furthermore, the Asphalt Concrete mix with modifier produced from 60/70 and 40/50 binder grades have lower flow than the Asphalt Concrete mix without modifier produced from 60/70 and 40/50 binder grades respectively. The addition of modifier in asphalt cement increases the viscosity consequently decreases the marshal flow (Chen et al, 2002).

Though the addition of modifier in the AC mix improved the marshal flow of both mixes with 40/50 and 60/70 binder grades, the level of improvement was different i.e. when modifier was added the marshal flow of mix with 60/70 binder grade decreased more than the mix with 40/50 binder grade.

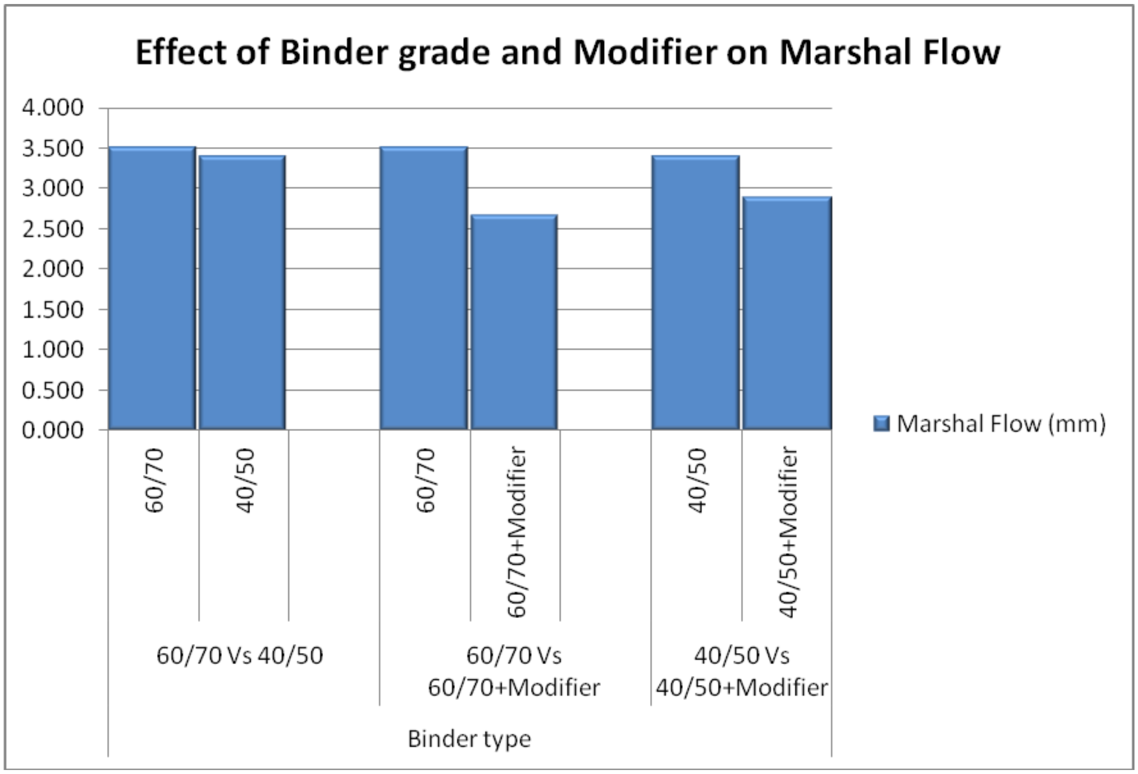

Figure 4.2 Effect of Binder grade and Modifier on Marshal Flow

Effect on Air Void

As shown in the figure 4.3 the AC mix produced from 40/50 binder grade has lower air void than the Asphalt concrete mix produced from 60/70 binder grade. Furthermore, the Asphalt Concrete mix with modifier produced from 60/70 and 40/50 binder grades have lower air void than the Asphalt Concrete mix without modifier produced from 60/70 and 40/50 binder grades respectively.

Even though the air void of asphalt concrete mix decreased with the addition of modifier, the decrease in air void was not significant when modifier was added in the asphalt concrete mix with 40/50 binder grade.

Generally, the volumetric properties, marshal stability and flow AC mixes were improved with the addition of modifier and/or when 40/50 binder grade was used instead of 60/70. Although these properties are not a direct measure of pavement performance, their improvement can improve the performance of pavement to some extent (Hafeez, 2009).

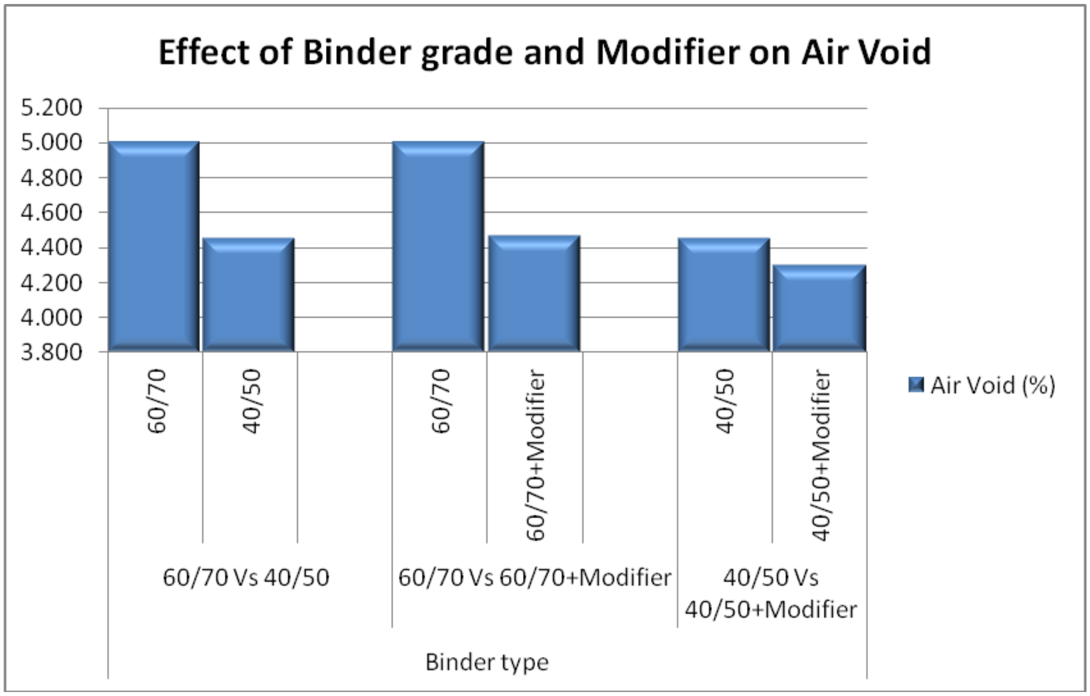

Figure 4.3 Effect of Binder grade and Modifier on Air Void

4.2 QZDCZSYY TEST RESULTS

Four specimens for each mix type which were made in the laboratory were tested in the QZDCZSYY to check for rutting potential. The rut depth and dynamic stability are presented in Tables 4.2. According to the Chinese specification, the minimum dynamic stability of AC mix for the QZDCZSYY test is 3000 time/mm. The dynamic stability of the mix with 60/70 binder grade without modifier was 1087 times/mm which is quite less than the required, 3000 times/mm i.e. the mix was highly rut susceptible and could not be used based on the specification. The complete rut depth and dynamic stability are provided in Appendix B. Analysis was done between AC mixes produced from the two binder grades and between mixes with and without modifier.

Figure 4.4 is an example of the sample after it has been tested in the QZDCZSYY. Other examples are presented in Appendix C.

Figure 4.4 Sample After QZDCZSYY Testing, 60/70 Binder Grade

Table 4.2 shows the mix with 60/70 binder grade had an average 60 minute rut depth of 4.36mm while the mix with 40/50 binder grade had an average 60 minute rut depth of 2.32mm. The rut depth was reduced by 47% when 40/50 binder grade was used instead of 60/70 according to the result. Furthermore, the dynamic stability of the mix with 40/50 binder grade is 3.5 times greater than the mix with 60/70 binder grade.

The mix with 60/70 binder grade and modifier had an average 60 minute rut depth of 1.50mm while the mix with 60/70 binder grade without modifier had an average 60 minute rut depth of 4.36mm. The percent change in average rut depth was 66% with the addition of modifier in the mix with 60/70 binder grade. In addition, the dynamic stability of the mix with 60/70 binder grade and modifier is about 7 times greater than the mix with 60/70 binder grade without modifier.

The mix with 40/50 binder grade and modifier had an average 60 minute rut depth of 0.75mm while the mix with 40/50 binder grade and without modifier had an average 60 minute rut depth of 2.32mm. The percent change in average rut depth was 68% with the addition of modifier in the mix with 40/50 binder grade. Besides, the dynamic stability of the mix with 40/50 binder grade and modifier is 3 times greater than the mix with 40/50 binder grade without modifier.

Table 4.2 Rut Values for Different Binder Grades and Modifier

Binder grade used	Sample No	45min Rutting (Deformation) (mm)	60min Rutting (Deformation) (mm)	**Average 60min Rutting (Deformation) (mm)**	Dynamic Stability (Times/mm)	**Average Dynamic Stability (Times/mm)**
60/70	1	4.483	5.247	**4.36**	825	**1087**
	2	3.697	4.213		1221	
	3	3.525	4.139		1026	
	4	3.354	3.847		1278	
60/70+Modifier (NES)	1	1.447	1.519	**1.50**	8750	**7427**
	2	2.526	2.615		7079	
	3	0.99	1.065		8400	
	4	0.699	0.814		5478	
40/50	1	1.875	1.985	**2.32**	5727	**3887**
	2	2.081	2.318		2658	
	3	2.092	2.297		3073	
	4	2.541	2.695		4091	
40/50+Modifier (NES)	1	0.325	0.38	**0.75**	11455	**11506**
	2	0.503	0.554		12353	
	3	0.969	1.022		11887	
	4	0.973	1.034		10328	

4.3 DATA ANALYSIS

The process of formulating the possible conclusions one can draw from an experiment and choosing between two alternatives is known as hypothesis testing. The null hypothesis, denoted by Ho, is the claim that is initially assumed to be true (the "prior belief" claim). The alternative hypothesis, denoted by Ha, is the assertion that is contradictory to Ho. The null hypothesis will be rejected in favor of the alternative hypothesis only if sample evidence suggests that Ho is false. If the sample does not strongly contradict Ho, we will continue to believe in the plausibility of the null hypothesis. The two possible conclusions from a hypothesis-testing analysis are then reject Ho or fail to reject Ho. A test of hypotheses is a method for using sample data to decide whether the null hypothesis should be rejected.

In order to examine the effect of binder and modifier on the rut depth, two samples with unknown variance t- tests were conducted. Since the objective of the experiment is to compare the effect of binders and modifier on mean rut depth, the parameter of interest is $\mu_1 - \mu_2$, the difference between the true average rut depths.

The null hypothesis for the test has been assumed as $\mu_1 - \mu_2 = 0$. The details of the t-test are given below:

Null Hypothesis: (4.1)

H_o: $\mu_1 - \mu_2 = 0$

Alternative Hypothesis;

H_a: $\mu_1 - \mu_2 > 0$ (4.2)

Test Statistic:

$$t = \frac{\bar{X}_1 - \bar{X}_2 - (\mu_1 - \mu_2)}{\sqrt{\left((n_1-1)*s_1^2\right)+\left((n_2-1)*s_2^2\right)}}\sqrt{\frac{n_1*n_2*(n_1+n_2-2)}{n_1+n2}} \qquad (4.3)$$

Where,

$\bar{x}_1$ and $\bar{x}_2$ are sample means

μ_1 and μ_2 are true means

$s_1^{\ 2}$ and s_2^2 are sample variances

n_1 and n_2 are the number of observations

The t-test function in Microsoft's Excel spread sheet or equation 4.3 was used to perform the Student t analysis. This t-test form assumes samples with unequal variance; it is referred to as a heteroscedastic t-test. The value returned by the function, P-value, is the probability that the two means of the samples are equal (Dekking et al, 2005).

By comparing the computed probability to the desired significance level, the null hypothesis can be either accepted or rejected. In traditional practices, a significance level of 5 percent was selected. However, demanding more compelling evidence, a significance level of 1 percent was selected for this analysis. Table 4.4 presents the P-values for the different laboratory samples.

Rejection and acceptance region: If the probability is less than the significance level, the null hypothesis should be rejected. On the contrary, the null hypothesis should be accepted (fail to reject the null hypothesis) when the probability is not less than the significance level (Devore, 2010).

Table 4.3 shows that the P-value for the comparison of the mixes with 60/70 and 40/50 binder grades is 0.3% which is less than 1 percent, indicating the null hypotheses should be rejected. This supports a conclusion that the rutting potential of these mixes is significantly different. The mix with 40/50 binder grade rutted less than the mix with 60/70 binder grade. Therefore, the average rutting potential for the mix produced with 40/50 binder grade was less than the mix produced from 60/70 binder.

The P-value for the comparison of the mixes produced with 60/70 binder grade without modifier and 60/70 binder grade with modifier is 0.16% which is far less than 1 percent as shown on Table 4.3. This indicates the null hypotheses should be rejected. This supports a conclusion that the rutting potential of these mixes is significantly different. The mix produced with 60/70 binder grade and modifier rutted less than the mix produced with 60/70 binder grade and without modifier.

Table 4.3 also shows that the P-value for the comparison of the mixes produced with 40/50 binder grade without modifier and 40/50 binder grade with modifier is

0.04% which is far less than 1 percent, indicating the null hypotheses should be rejected. This supports a conclusion that the rutting potential of these mixes is significantly different. The average rutting potential for the mix produced with 40/50 binder grade and modifier was less than the mix produced with 40/50 binder grade and without modifier. For both 60/70 and 40/50 binder grades, the mix with modifier rutted less than without modifier leading to a conclusion that the rutting potential is reduced with the use of the modifier.

Table 4.3 P-Values for Different Mix Type

Comparison	60/70 Vs 40/50		60/70 Vs 60/70+Modifier		40/50 Vs 40/50+Modifier	
Binder Used	60/70	40/50	60/70	60/70+Modifier	40/50	40/50+Modifier
Sample Rut depth(mm)	5.247	1.985	5.247	1.519	1.985	0.38
	4.213	2.318	4.213	2.615	2.318	0.554
	4.139	2.297	4.139	1.065	2.297	1.022
	3.847	2.695	3.847	0.814	2.695	1.034
Sample Mean	4.362	2.324	4.362	1.503	2.324	0.748
Sample Variance	0.373	0.084	0.373	0.634	0.084	0.110
t-value	6.023		5.694		7.150	
p-value	**0.0030**		**0.0016**		**0.0004**	

The following chart also shows the comparison between rut depth and dynamic stability of different mixes i.e. 60/70 Vs 40/50, 60/70 Vs 60/70 with modifier and 40/50 Vs 40/50 with modifier.

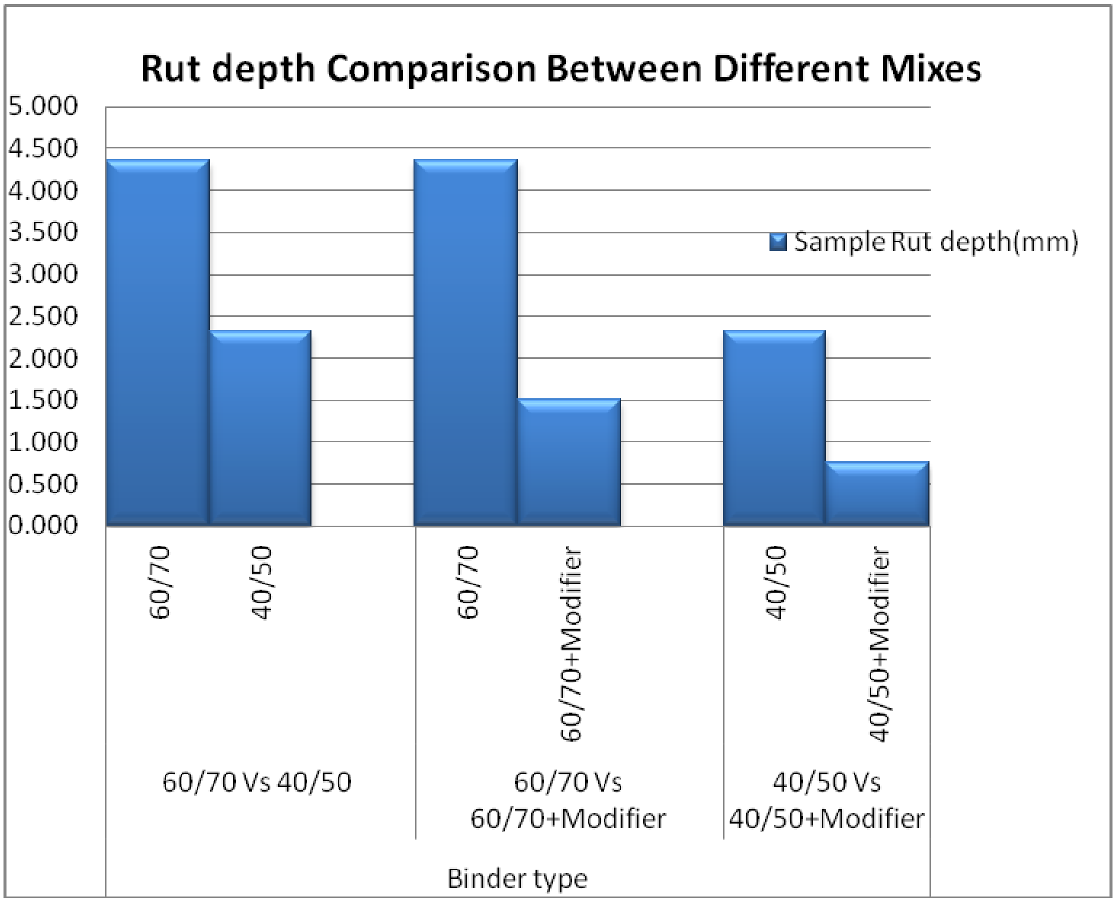

Figure 4.5 Comparison of the Rut Depth of Mixes Based on the Binder Grade and Modifier Used

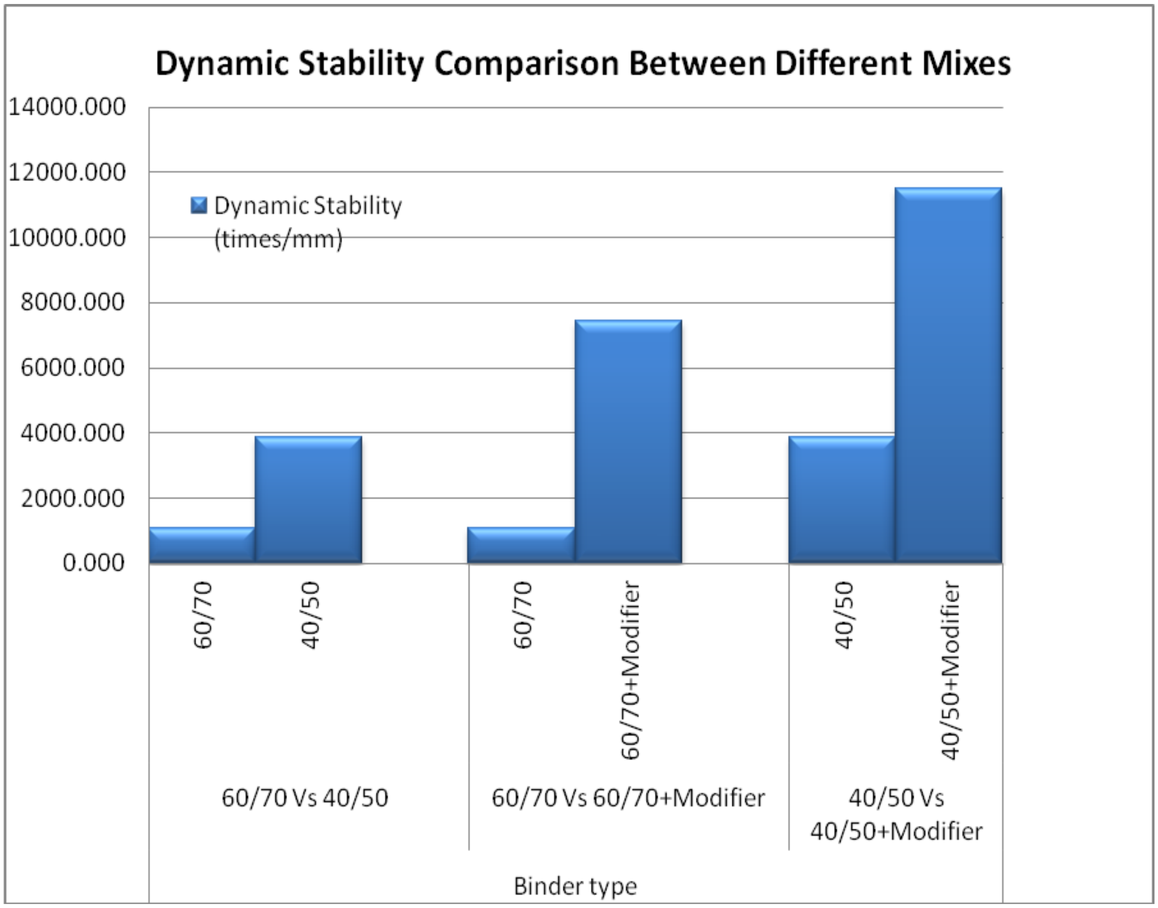

Figure 4.6 Comparison of the Dynamic Stability of Mixes Based on the Binder Grade and Modifier Used

4.4 EFFECT OF MODIFIERS ON OTHER PERFORMANCE PROPERTIES

The absorption and anti-stripping ability of modified asphalt is stronger than base asphalt. Modified binders have higher softening point, higher viscosity and lower penetration thus improving high temperature performance. Furthermore modifiers can effectively improve the surface adhesion of aggregate. Due to the mentioned reasons the asphalt concrete mix with modifiers performs better (W. Huang and G. Xu, 2012).

Modified binders exhibit higher fatigue lives in comparison to base binders. The fatigue lives of modified binder mixes are generally longer than those unmodified binder mixes, indicating that mixes with binder have higher resistance to fracture and fatigue. In addition, the cracking resistance of binder increases with the presence of modifier. (Hrdlicka et al, 2007)

CHAPTER 5 CONCLUSIONS AND RECOMMENDATIONS

5.1 CONCLUSIONS

During the past several years many roads in Ethiopia experienced problems of permanent deformation, or rutting, in hot mix asphalt pavements. Among these roads, Goha Tsion – Dejen Road and Adama – Metehara Road can be mentioned. In Goha Tsion – Dejen Road, rutting was noticed on considerable length of the pavement of the road after two months of construction. As shown in the figure 1.1 the problem was aggravated during service time after the whole road was opened for traffic.

Due to the seriousness of the problem of rutting in the road construction sector of the country it was decided to perform a laboratory evaluation of the rutting potential of different asphalt concrete mixes. In particular, the evaluation was based on asphalt concrete mixes produced with different binder grades and modifiers. The laboratory evaluation using the QZDCZSYY allowed a good means for comparing the relative performance of these mixes.

Based on the literature review, testing and analysis of test results of this research work, the following conclusions were made.

- The mixes with 40/50 binder grade have a lower rutting potential (perform better) than the mixes with 60/70 binder grade.

- The mixes produced from 60/70 binder grade with modifier have a lower rutting potential (perform better) than the mixes with 60/70 binder grade without modifier.

- The mixes produced from 40/50 binder grade with modifier have a lower rutting potential (perform better) than the mixes with 40/50 binder grade without modifier.

- The indications of this research are that the mixes with better binder grade have a lower rutting potential and also the use of modifiers in asphalt concrete mix appears to provide mixes with superior performance.

- Higher marshal stability values were obtained from mixtures with 40/50 binder grade rather than 60/70 binder grade.

- The air void was lower when 40/50 binder grade was used instead of 60/70 binder grade.

- The air void and marshal flow decreases when modifiers are added in asphalt concrete mixes.

- The bulk density increases with the addition of modifier in asphalt concrete mix.

- Higher bulk density values were obtained from mixtures with 40/50 binder grade rather than 60/70 binder grade.

5.2 RECOMMENDATIONS

Measuring the rutting performance of the asphalt concrete mix is valuable for the purpose of the right binder grade selection, asphalt concrete mix design and selection which are very important for the long and good service of an asphalt surface. Therefore, the rutting performance of the asphalt concrete mix should be measured before the commencement of asphalt surface construction.

This research indicates that the use of modifiers in asphalt concrete mix appears to provide mixes with superior performance. The use of modifiers should be considered at rut susceptible locations which include high temperature areas, intersections with high truck volume and other locations with heavy traffic and slow speeds, such as truck weigh stations and climbing lanes.

The addition of modifier in asphalt concrete mixes improves the volumetric properties, marshal stability and flow of the mix. Therefore, the use of modifiers should be considered for mixes which fails to meet the criteria of volumetric properties, marshal stability and flow.

5.3 LIMITATIONS AND ISSUES FOR FUTURE STUDY

This research was limited to a single source of modifier which is NES. As demonstrated in the literature review, there are a wide variety of modifiers available in the world market. The results produced during this research should not be used to infer that other products would perform equally well. Further research should be performed with alternative suppliers of modifier.

Although the 40/50 binder grade used in this study appears to provide better performance than 60/70 binder grade, it is more expensive. And also the modifier used in this study appears to provide superior performance; it is more expensive than unmodified materials. The decision to use the more expensive material should be based on the design period cost benefit analysis or other economic feasibility tools. Therefore, economic feasibility study should be conducted to evaluate the alternatives.

One of the limitations of this study was that the decision was made to use the mix design asphalt contents for the evaluation of the other binder types. Actually the volumetric properties of the mix for both binder types were checked and met the criteria. Taking the same mix design asphalt contents provided consistency in the treatment of the mixes. An alternative to this decision would be to determine the optimum asphalt content for each binder and use the revised mix design for the evaluation.

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APPENDIX A: MIX PROPERTIES

Table A.1 Theoretical maximum specific gravities

Table A.1 Theoretical Maximum Specific Gravities

Binder Used	Asphalt/Aggregate (%)	Wt of dry sample in air (g)	Wt of vessel filled with water (g)	Wt of vessel filled with water and sample at 25℃ (g)	MTD	Average
60/70	4.8	2538	6595	8172	2.641	2.641
	4.8	2541	6596	8175	2.641	
60/70+Modifier	4.8	1562	6593	7564	2.643	2.643
	4.8	1567	6594	7568.2	2.643	
40/50	4.8	2532	6595	8170	2.646	2.644
	4.8	2534	6596	8171	2.642	
40/50+Modifier	4.8	1525	6593	7541	2.643	2.644
	4.8	1527	6594	7543.5	2.644	

Table A.2 Standard stability, flow and volumetric properties

Table A.2 Test Record of The Standard Stability, Flow and Volumetric Properties of AC Mix with Binder Grade of 60/70

Binder Grade	60/70	Modifier Used	-	Kind of Mixture	AC-13 for Wearing Course

Kinds of Material	10-16mm	5-10mm	3-5mm	0-3	Filler	Asphalt		Size of Mould (mm)	ϕ101.6*63.5
Apparent specific	2.978	2.992	2.806	2.999	2.866	1.024		Tamping No (n)	75
Bulk specific gravity	2.832	2.839	2.591	2.772				Tamping Temperature (^{o}C)	180
Water absorption (%)	1.72	1.80	2.96	2.74				Asphalt/Aggregate (%)	4.8
Material Proportion (%)	29.0	30.0	3.0	37.0	1.0	4.8		Asphalt Content (%)	4.58
Trial Mix (g)	1740.0	1800.0	180.0	2220.0	60.0	288.0		Measured MTD	2.641

NO.	Sample Size (mm)						Wt in Air (g)	Wt in Water (g)	Wt of SSD (g)	Bulk Denity	VV (%)	VMA (%)	VFA (%)	Stability (KN)	Flow value (0.1mm)
	Dia (mm)	Height (mm)													
		1	2	3	4	Average									
1	101.6	63.1	63.2	62.9	63.0	63.1	1250.5	755.5	1251.9	2.519	4.6	14.2	67.6	19.00	35.0
2	101.6	63.4	63.3	63.1	63.2	63.3	1246.8	750.6	1248.9	2.502	5.3	14.8	64.5	13.70	37.1
3	101.6	63.7	63.7	63.6	63.8	63.7	1249.8	753.3	1251.5	2.509	5.0	14.6	65.7	14.70	37.8
4	101.6	63.4	63.7	63.6	63.4	63.5	1240.2	746.0	1241.4	2.503	5.2	14.8	64.8	14.99	30.8
Average										**2.508**	**5.0**	**14.6**	**65.7**	**15.60**	**35.2**
Remarks															

Table A.3 Test Record of The Standard Stability, Flow and Volumetric Properties of AC Mix with Binder Grade of 60/70 and NES

Binder Grade	60/70	Modifier Used	HSA(NES-1)	Kind of Mixture	AC-13 for Wearing Course

Kinds of Material	10-16mm	5-10mm	3-5mm	0-3	Filler	Asphalt	Modifier HSA(NES-1)	Size of Mould (mm)	ɸ101.6*63.5
Apparent specific	2.978	2.992	2.806	2.999	2.866	1.024	0.906	Tamping No (n)	75
Bulk specific gravity	2.832	2.839	2.591	2.772				Tamping Temperature (^oC)	180
Water absorption (%)	1.72	1.80	2.96	2.74				Asphalt/Aggregate (%)	4.8
Material Proportion (%)	29.0	30.0	3.0	37.0	1.0	4.8	0.4	Asphalt Content (%)	4.58
Trial Mix (g)	1740.0	1800.0	180.0	2220.0	60.0	288.0	26.00	Measured MTD	2.643

NO.	Sample Size (mm)						Wt in Air (g)	Wt in Water (g)	Wt of SSD (g)	Bulk Denity	VV (%)	VMA (%)	VFA (%)	Stability (KN)	Flow value (0.1mm)
	Dia (mm)	Height (mm)													
		1	2	3	4	Average									
1	101.6	63.6	63.2	62.9	63.0	63.2	1247.3	755.3	1248.6	2.528	4.3	13.9	68.9	16.10	24.4
2	101.6	62.8	62.7	62.6	62.9	62.8	1244.3	749.8	1245.4	2.511	5.0	14.5	65.6	16.10	26.7
3	101.6	63.0	63.1	62.8	63.2	63.0	1251.4	754	1252.3	2.511	5.0	14.5	65.7	16.40	27.8
4	101.6	63.3	63.1	63.2	63.4	63.3	1246.3	753.7	1247.1	2.526	4.4	14.0	68.4	18.30	27.8
4	101.6	62.7	62.8	62.9	63.0	62.9	1245.5	760.2	1251.2	2.537	4.0	13.7	70.5	15.60	25.8
5	101.6	63.1	63.5	63.7	63.4	63.4	1243.3	756.8	1247.2	2.535	4.1	13.7	70.2	15.90	27.8
Average										**2.525**	**4.47**	**14.1**	**68.2**	**16.40**	**26.7**
Remarks															

Table A.4 Test Record of The Standard Stability, Flow and Volumetric Properties of AC Mix with Binder Grade of 40/50

Binder Grade	40/50	Modifier Used	-	Kind of Mixture	AC-13 for Wearing Course

Kinds of Material	10-16mm	5-10mm	3-5mm	0-3	Filler	Asphalt		Size of Mould (mm)	ϕ101.6*63.5
Apparent specific	2.978	2.992	2.806	2.999	2.866	1.024		Tamping No (n)	75
Bulk specific gravity	2.832	2.839	2.591	2.772				Tamping Temperature (^{o}C)	180
Water absorption (%)	1.72	1.80	2.96	2.74				Asphalt/Aggregate (%)	4.8
Material Proportion (%)	29.0	30.0	3.0	37.0	1.0	4.8		Asphalt Content (%)	4.58
Trial Mix (g)	1740.0	1800.0	180.0	2220.0	60.0	288.0		Measured MTD	2.644

NO.	Sample Size (mm)						Wt in Air (g)	Wt in Water (g)	Wt of SSD (g)	Bulk Denity	VV (%)	VMA (%)	VFA (%)	Stability (KN)	Flow value (0.1mm)
	Dia (mm)	Height (mm)													
		1	2	3	4	Average									
1	101.6	63.3	63.4	63.1	63.2	63.3	1251.8	757.6	1253.4	2.525	4.5	14.1	67.9	13.30	36.0
2	101.6	63.4	63.1	63.2	63.0	63.2	1256.8	763.3	1258.2	2.540	4.0	13.6	70.8	16.61	37.5
3	101.6	62.9	62.9	63.4	63.3	63.1	1250.9	756.8	1252.3	2.525	4.5	14.1	67.9	14.76	29.7
4	101.6	63.0	62.9	63.1	63.0	63.0	1242.3	751.8	1243.4	2.527	4.4	14.0	68.4	18.16	35.8
5	101.6	62.7	62.5	62.4	62.5	62.5	1239.1	747.8	1240.3	2.516	4.8	14.4	66.3	17.88	34.7
Average										**2.526**	**4.45**	**14.0**	**68.3**	**16.14**	**34.7**
Remarks															

Table A.5 Test Record of The Standard Stability, Flow and Volumetric Properties of AC Mix with Binder Grade of 40/50 and Modifier

Binder Grade	40/50	Modifier Used	HSA(NES-1)	Kind of Mixture	AC-13 for Wearing Course

Kinds of Material	10-16mm	5-10mm	3-5mm	0-3	Filler	Asphalt	Modifier HSA(NES-1)	Size of Mould (mm)	ϕ101.6*63.5
Apparent specific	2.978	2.992	2.806	2.999	2.866	1.024	0.906	Tamping No (n)	75
Bulk specific gravity	2.832	2.839	2.591	2.772				Tamping Temperature (^{o}C)	180
Water absorption (%)	1.72	1.80	2.96	2.74				Asphalt/Aggregate (%)	4.8
Material Proportion (%)	29.0	30.0	3.0	37.0	1.0	4.8	0.4	Asphalt Content (%)	4.58
Trial Mix (g)	1740.0	1800.0	180.0	2220.0	60.0	288.0	26.00	Measured MTD	2.644

NO.	Sample Size (mm)						Wt in Air (g)	Wt in Water (g)	Wt of SSD (g)	Bulk Denity	VV (%)	VMA (%)	VFA (%)	Stability (KN)	Flow value (0.1mm)
	Dia (mm)	Height (mm)													
		1	2	3	4	Average									
1	101.6	62.8	62.6	62.4	62.7	62.6	1237	764.0	1254.5	2.522	4.6	14.2	67.4	16.80	36.7
2	101.6	62.9	63.1	63.0	62.9	63.0	1241.9	761	1251.8	2.530	4.3	13.9	69.0	17.20	27.8
3	101.6	63.1	63.2	63.2	63.6	63.3	1249.9	758	1251.5	2.533	4.2	13.8	69.5	18.50	26.4
4	101.6	62.9	62.7	63.1	62.9	62.9	1244.1	763.4	1254.6	2.533	4.2	13.8	69.5	18.50	27.8
5	101.6	63.3	63.1	63.4	63.0	63.2	1249	758.1	1251.3	2.532	4.2	13.8	69.4	17.70	26.9
5	101.6	62.8	62.5	62.6	62.7	62.7	1242	761.2	1252.8	2.526	4.4	14.0	68.2	16.90	27.8
Average										**2.529**	**4.33**	**14**	**68.8**	**17.60**	**28.9**

Remarks	

APPENDIX B

RUT DEPTHS AND DYNAMIC STABILITY

Table B.1 Test Record of Deformation(Rutting) of AC Mix with 60/70

Proposed Use	Wearing Course	Binder Grade	60/70	Modifier Used	-
Temperature of Test	60^{o}C	Pressure of Wheel Roller	0.7Mpa	Rolling Speed	42 Times/Min

Kinds of Material	10-16mm	5-10mm	3-5mm	0-3	Filler	Asphalt		Size of Mould (mm)	300*300*50mm
Apparent specific gravity	2.978	2.992	2.806	2.999	2.866	1.024		Wheel Roller No (n)	18
Bulk specific gravity	2.832	2.839	2.591	2.772				Mould Temperature (^{o}C)	170
Water absorption (%)	1.72	1.80	2.96	2.74				Asphalt/Aggregate (%)	4.8
Material Proportion (%)	29.0	30.0	3.0	37.0	1.0	4.8		Asphalt Content (%)	4.58
Trial Mix (g)	1740.0	1800.0	180.0	2220.0	60.0	288.0		Measured MTD	2.641

Sample No	45min Rutting (Deformation) (mm)	60min Rutting (Deformation) (mm)	Dynamic Stability (Times/mm)	**Average 60min Rutting (Deformation) (mm)**	**Average Dynamic Stability (Times/mm)**	Remarks
1	4.483	5.247	825	**4.36**	**1087**	
2	3.697	4.213	1221			
3	3.525	4.139	1026			
4	3.354	3.847	1278			

Table B.2 Test Record of Deformation(Rutting) of AC Mix with 60/70 and Modifier

Proposed Use	Wearing Course	Binder Grade	60/70	Modifier Used	HSA(NES-1)
Temperature of Test	60^{o}C	Pressure of Wheel Roller	0.7Mpa	Rolling Speed	42 Times/Min

Kinds of Material	10-16mm	5-10mm	3-5mm	0-3	Filler	Asphalt	Modifier HSA(NES-1)	Size of Mould (mm)	300*300*50mm
Apparent specific	2.978	2.992	2.806	2.999	2.866	1.024	0.906	Wheel Roller No (n)	18
Bulk specific gravity	2.832	2.839	2.591	2.772				Mould Temperature (^{o}C)	170
Water absorption (%)	1.72	1.80	2.96	2.74				Asphalt/Aggregate (%)	4.8
Material Proportion (%)	29.0	30.0	3.0	37.0	1.0	4.8	0.4	Asphalt Content (%)	4.58
Trial Mix (g)	1740.0	1800.0	180.0	2220.0	60.0	288.0	26.00	Measured MTD	2.643

Sample No	45min Rutting (Deformation) (mm)	60min Rutting (Deformation) (mm)	Dynamic Stability (Times/mm)	**Average 60min Rutting (Deformation) (mm)**	**Average Dynamic Stability (Times/mm)**	Remarks
1	1.447	1.519	8750	**1.50**	**7427**	
2	2.526	2.615	7079			
3	0.99	1.065	8400			
4	0.699	0.814	5478			

Table B.3 Test Record of Deformation(Rutting) of AC Mix with 40/50

Proposed Use	Wearing Course	Binder Grade	40/50	Modifier Used	-
Temperature of Test	60°C	Pressure of Wheel Roller	0.7Mpa	Rolling Speed	42 Times/Min

Kinds of Material	10-16mm	5-10mm	3-5mm	0-3	Filler	Asphalt		Size of Mould (mm)	300*300*50mm
Apparent specific	2.978	2.992	2.806	2.999	2.866	1.024		Wheel Roller No (n)	18
Bulk specific gravity	2.832	2.839	2.591	2.772				Mould Temperature (°C)	170
Water absorption (%)	1.72	1.80	2.96	2.74				Asphalt/Aggregate (%)	4.8
Material Proportion (%)	29.0	30.0	3.0	37.0	1.0	4.8		Asphalt Content (%)	4.58
Trial Mix (g)	1740.0	1800.0	180.0	2220.0	60.0	288.0		Measured MTD	2.644

Sample No	45min Rutting (Deformation) (mm)	60min Rutting (Deformation) (mm)	Dynamic Stability (Times/mm)	**Average 60min Rutting (Deformation) (mm)**	**Average Dynamic Stability (Times/mm)**	Remarks
1	1.875	1.985	5727	**2.32**	**3887**	
2	2.081	2.318	2658			
3	2.092	2.297	3073			
4	2.541	2.695	4091			

Table B.4 Test Record of Deformation(Rutting) of AC Mix with 40/50 and Modifier

Proposed Use	Wearing Course	Binder Grade	40/50	Modifier Used	HSA(NES-1)
Temperature of Test	60°C	Pressure of Wheel Roller	0.7Mpa	Rolling Speed	42 Times/Min

Kinds of Material	10-16mm	5-10mm	3-5mm	0-3	Filler	Asphalt	Modifier HSA(NES-1)	Size of Mould (mm)	300*300*50mm
Apparent specific	2.978	2.992	2.806	2.999	2.866	1.024	0.906	Wheel Roller No (n)	18
Bulk specific gravity	2.832	2.839	2.591	2.772				Mould Temperature (°C)	170
Water absorption (%)	1.72	1.80	2.96	2.74				Asphalt/Aggregate (%)	4.8
Material Proportion (%)	29.0	30.0	3.0	37.0	1.0	4.8	0.3	Asphalt Content (%)	4.58
Trial Mix (g)	1740.0	1800.0	180.0	2220.0	60.0	288.0	19.00	Measured MTD	2.644

Sample No	45min Rutting (Deformation) (mm)	60min Rutting (Deformation) (mm)	Dynamic Stability (Times/mm)	**Average 60min Rutting (Deformation) (mm)**	**Average Dynamic Stability (Times/mm)**	Remarks
1	0.325	0.38	11455	**0.75**	**11506**	
2	0.503	0.554	12353			
3	0.969	1.022	11887			
4	0.973	1.034	10328			

APPENDIX C

SAMPLE TEST RESULTS

沥青混合料车辙试验报告

试件编号:40/50 NO1

试验温度: 60 ℃

试件尺寸:

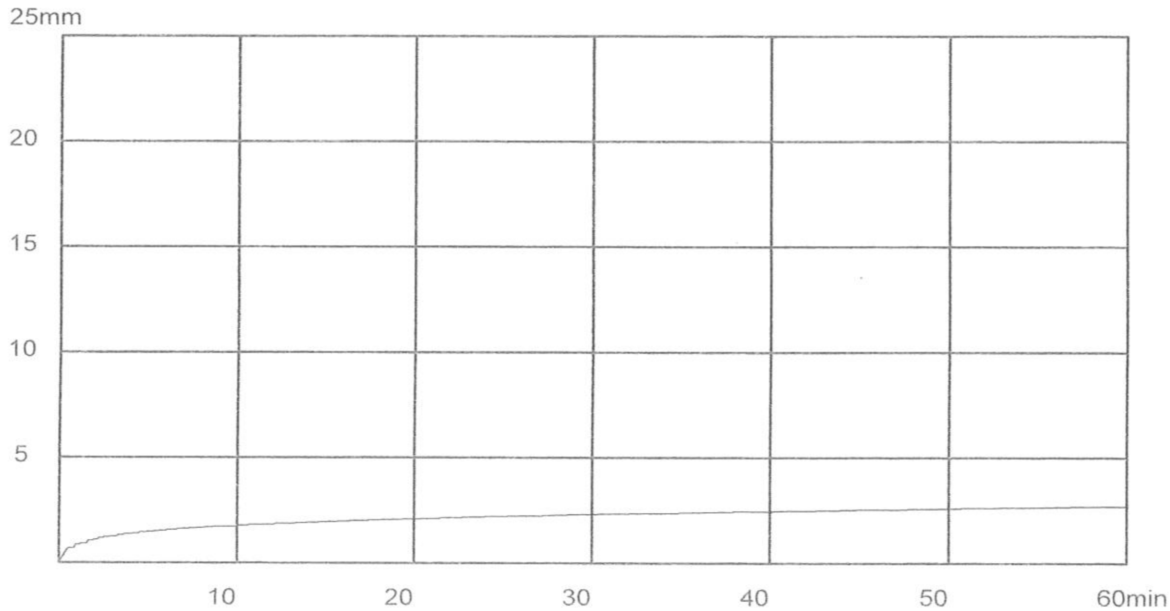
试轮接触压强:0.7MPa

试轮行走距离:230±10mm

试轮行走速度: 42±1 次/min

试验日期:2013-8-29

车辙试验变形曲线



d1= 2.541 mm

d2 = 2.695 mm

t1= 45 min

t2= 60 min

C1=1.0

C2= 1

沥青混合料试件的动稳定度为:

DS=(t2-t1)*42/(d2-d1)*C1*C2= 4090.909

沥青混合料车辙试验报告

试件编号:60/70+NES

试验温度: 60 ℃

试件尺寸:

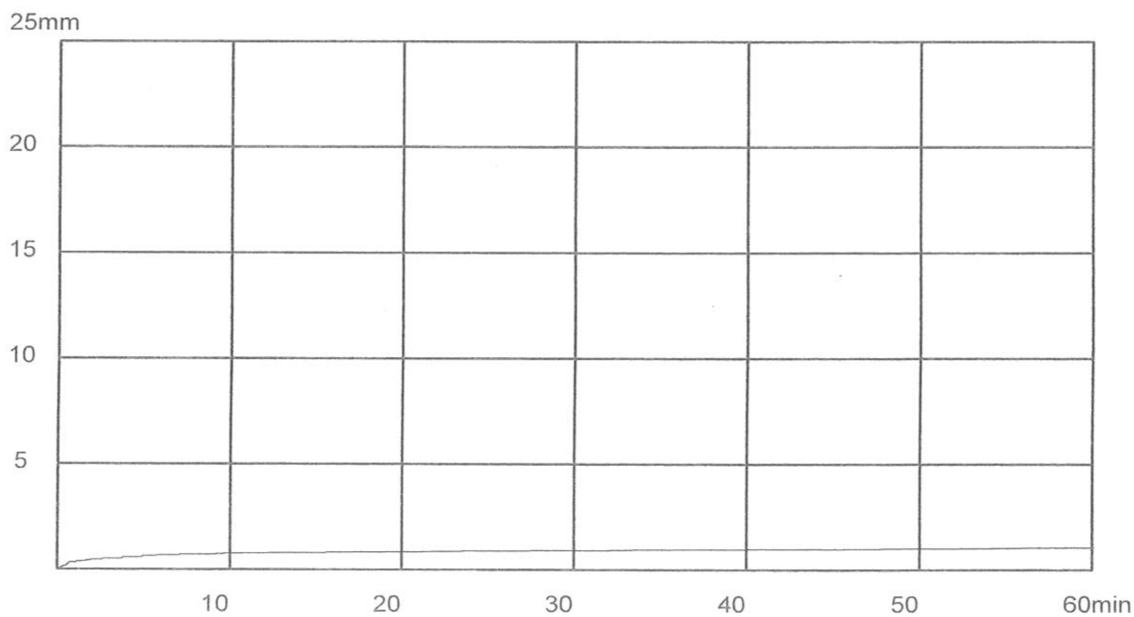
试轮接触压强:0.7MPa

试轮行走距离:230±10mm

试轮行走速度: 42±1 次/min

试验日期:2014-3-5

车辙试验变形曲线



d1= .99 mm

d2 = 1.065 mm

t1= 45 min

t2= 60 min

C1=1.0

C2= 1

沥青混合料试件的动稳定度为:

DS=(t2-t1)*42/(d2-d1)*C1*C2= 8400

沥青混合料车辙试验报告

试件编号:40/50 NO1

试验温度: 60 ℃

试件尺寸:

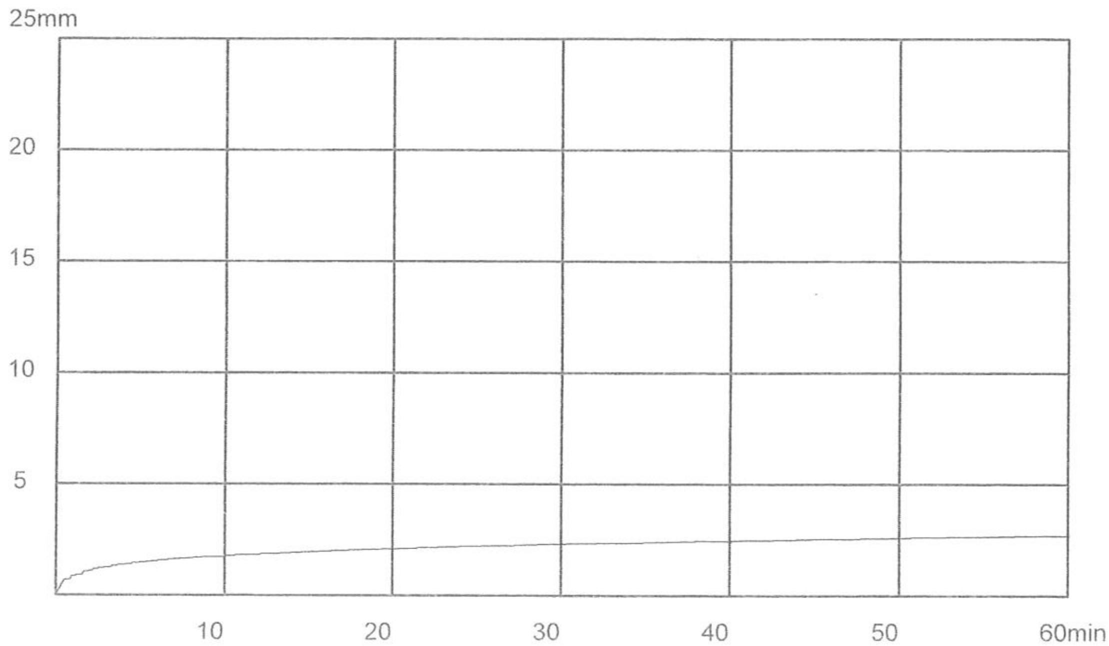
试轮接触压强:0.7MPa

试轮行走距离:230±10mm

试轮行走速度: 42±1 次/min

试验日期:2013-8-29

车辙试验变形曲线



d1= 2.541 mm

d2 = 2.695 mm

t1= 45 min

t2= 60 min

C1=1.0

C2= 1

沥青混合料试件的动稳定度为:

DS=(t2-t1)*42/(d2-d1)*C1*C2= 4090.909

沥青混合料车辙试验报告

试件编号:40/50+NES

试验温度: 60 ℃

试件尺寸:

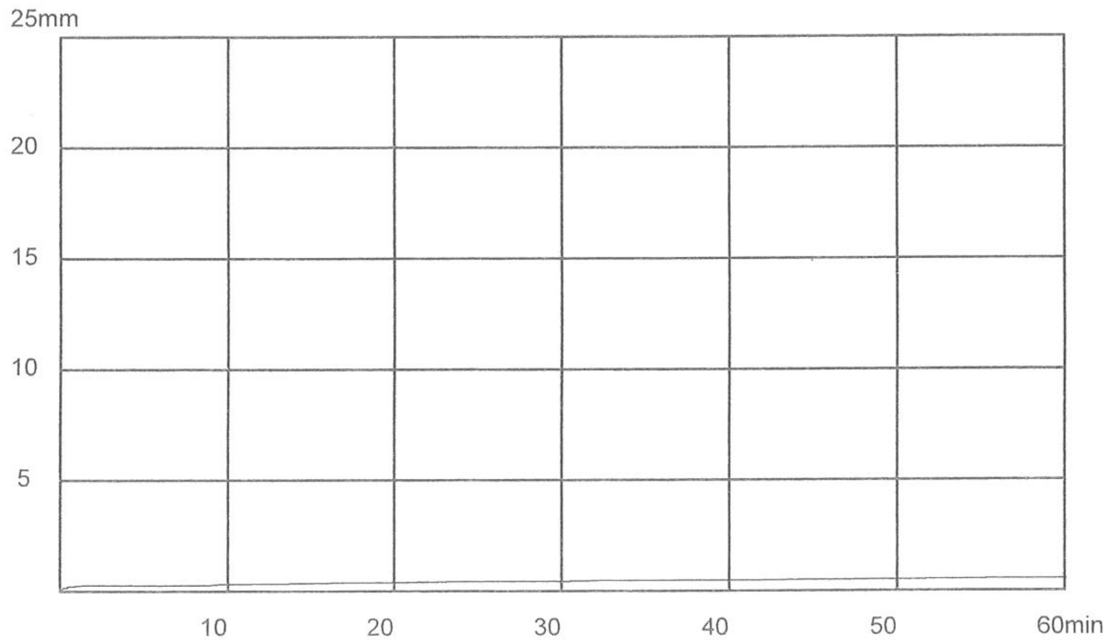
试轮接触压强:0.7MPa

试轮行走距离:230±10mm

试轮行走速度: 42±1 次/min

试验日期:2014-3-4

车辙试验变形曲线



d1= .503 mm

d2 = .554 mm

t1= 45 min

t2= 60 min

C1=1.0

C2= 1

沥青混合料试件的动稳定度为:

DS=(t2-t1)*42/(d2-d1)*C1*C2= 12352.94

APPENDIX D: PHOTOGRAPHS OF RUT DEPTH SPECIMENS

Figure D.1 Specimen with 60/70 Binder Grade

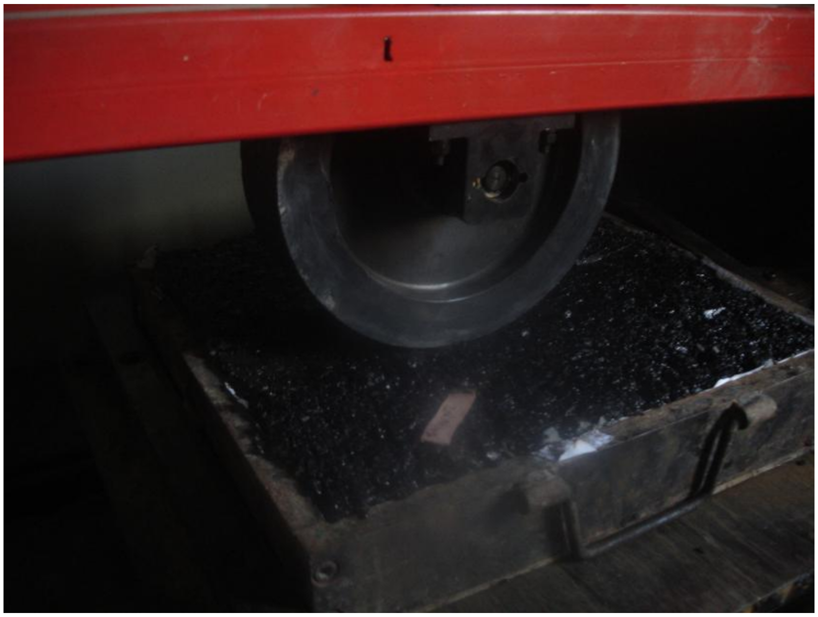


Figure D.2 Specimen with 60/70 Binder Grade and Modifier

Figure D.3 Specimen with 40/50 Binder Grade

Figure D.4 Specimen with 40/50 Binder Grade and Modifier