VERIFICATION OF APPROPRIATE TEMPERATURE FOR ASPHALT PAVEMENT LAYING: A CASE STUDY OF KADUNA STATE

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A DISSERTATION SUBMITTED TO THE SCHOOL OF POSTGRADUATE STUDIES,
AHMADU BELLO UNIVERSITY
ZARIA-NIGERIA,

IN PARTIAL FULFILLMENT OF THE REQUIREMENTS FOR THE AWARD MASTER OF SCIENCE DEGREE IN CIVIL ENGINEERING (M.Sc. (CIVIL).

DEPARTMENT OF CIVIL ENGINEERING
FACULTY OF ENGINEERING,
AHMADU BELLO UNIVERSITY
ZARIA-NIGERIA

JANUARY, 2016

DECLARATION

I hereby declare that this dissertation entitled **‘‘Verification of Appropriate Temperature for Asphalt Pavement Laying: A Case Study of Kaduna State’’** was carried out by me in the Department of Civil Engineering. The information derived from the literature has been duly acknowledged in the text and a list of references provided. No part of this dissertation was previously presented for another degree or diploma at this or any other institution.

Mohammed IbrahimUSMAN

Name of Student	Signature	Date

CERTIFICATION

This dissertation entitled **''VERIFICATION OF APPROPRIATE TEMPERATURE FOR ASPHALT PAVEMENT LAYING: A CASE STUDY OF KADUNA STATE''** by Mohammed Ibrahim USMAN, meets the regulations governing the award of degree of Master of Science in Civil Engineering Department of the Ahmadu Bello University, and is approved for contribution to knowledge and literacy presentation.

Dr. A. T. Olowosulu ______________________ ______________________ ______________________

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Dr. M. Joel ______________________ ______________________ ______________________

Member, Supervisory Committee Signature Date

Dr. Y. D. Amartey ______________________ ______________________ ______________________

Head of Department Signature Date

Prof. KabirBala ______________________ ______________________ ______________________

Dean, School of Postgraduate Studies Signature Date

DEDICATION

This dissertation is dedicated to the lovely memory of my late parent Alhaji Usman Abdullahi and Hajiya Fatima Mohammed. May their gentle soul rest in peace.

ACKNOWLEDGEMENT

Above all, I humbly thank Almighty Allah for seeing me through and teaching me to depend on Him always.I would like to express heartfelt thanks to a number of people who have supported and made this research possible. Firstly I would like to thank my academic supervisor Dr. A.T Olowosulu and Dr. M Joel for their commitment, support, patience and guidance which have been invaluable. My appreciation also goes to the H.O.D, Dr. Y.D Amartey for his endless encouragement. My appreciation goes to my family, for their patience and understanding throughout my study. Also to my associates both in school and my place of work, especially Engr. Ahmed Hadi Ashara and Engr Abdulmumin A Shuaibu to mention but a few. May the good Lord reward them in abundance.

ABSTRACT

This work is aimed at estimating the time available for compaction of a hot-mix asphalt mixture during construction.Laboratory tests were conducted on dense-graded asphalt and porous asphalt PA mixtures to evaluate their mechanical properties. Temperatures and mix characteristics tested in the laboratory were determined to be those typically found in an asphalt lift from initial lay down through final compaction. On the field, digital thermometers were installed at different depths to measure temperature changes to decide pavement cooling times of the same mixes as used in the laboratory. The cooling behavior of asphalt concrete could be classified into the following three stages: rapid, transition and stable zone. The air void in the PA mix was found to contribute to heat loss and result in a rapid cooling rate when compared to that in a dense-graded mix. The cooling rate at depth of 0, 2.5, and 5 cm showed an essential difference in the cooling rate initially. As the cooling process continued, the cooling rate became stable and reached a thermal equilibrium condition. An increase in lift thickness was shown to increase the compaction time. However, an increase in layer thickness more than 10 cm might not increase the time available for compaction significantly. A regression model was developed to predict the time required to cool to the minimum temperature allowed for compaction. The predicted values were in good agreement with those measured in the field. This study shows that a simplified method to predict available compaction time can be developed by considering only the significant factors affecting pavement cooling.

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LIST OF SYMBOLS/ABBREVIATION

HMA:	Hot Mix Asphalt
AASHTO:	American Association of State Highway and Transportation Officials
TOT:	Traffic Opening Time
MMAT:	Mean Monthly Average Temperatures
TMY:	Typical Metrological Year
LTPP:	Long Term Pavement Performance
LTPP-SMP:	Long Term Pavement Performance study's seasonal Monitoring Program
WMA:	Warm Mix Asphalt concrete
TTC:	Transport Technology Centre
NITT:	Institute of Transport Technology
FERMA:	Federal Road Maintenance Agency
NCAT:	Nigerian College of Aviation Technology
IAR:	Institute for Agricultural Research

CHAPTER ONE

INTRODUCTION

1.1 Preamble

Thermal environmental conditions, to which pavements continuously are exposed in the constructionand repair phases, as well as in use, determine the temperature profile in asphaltic sections. Fluctuations in ambient air temperatures - diurnal and seasonal, intensity of solar radiation, pavement materials and geometry, convective surface conditions,and precipitation significantly impact pavement stability and therefore the long-term success of pavement design. Accurate prediction of the temperature profile in pavement construction greatly aids pavement engineers,specifically in the assessment of pavement deflection, in calculations of pavement modulusvalues, in estimations of frost action and frost penetration and seasonal heatingand cooling effects.

The top pavement layer normally is exposed to greater temperature fluctuations than the layers below it. Because of this, detailed knowledge of the temperature distribution in asphalt layers also allows for a more sophisticated specification ofasphalt binders for lower lifts through specification of less expensive asphalt binders for lower lifts, and thus it provides an economical solution to rising pavement construction costs. An assessment of the impact of pavement temperature variations on various pavement materials, such as dense and open- graded asphalt mixes is possible with a higher degree of accuracy.

This include a grading system called performance grading (PG) that proposes atwo-number system intended to ensure that the proper asphalt binder is used to resist pavement rutting in hot temperatures and to resist cracking in cold temperatures. The approachrepresents the expected maximum high and low asphalt temperatures based on local climatic data for the hottest and coldest times of the year.

However, this raises questions with respect to pavement temperature estimations since performance grading method for asphalt binders appears to modify the asphalt operational temperature range, and thus further limits availability of asphalt that meets the prescribed criteria. One concern associated with is the cost, since both asphaltic cement and aggregate costs may be higher for pavement mixes due to limited sources or increased processing than for normal mix designs.

In this case, the cost of asphalt may increase by as much as 30 percent over conventional implementations. Thepavement performance grading requirements for lower asphalt lifts-including the binder and base courses, and the appropriate binder selection for hot mix asphalt recycling, callsfor a detailed understanding of the temperature profile in pavements construction. Many methods dealing with prediction of temperature gradients in pavements are based on statistical and probabilistic methods developed based on weather and pavement data collected through the Long-Term Pavement Performance.

However, such statistical and probabilistic methods display shortcomings in that they tend to either underestimate high pavement temperatures or overestimate low pavement temperatures raising questions about their accuracy and reliability. More detailed methods using energy balance equations to estimate pavement surface temperatures or numerical models that attempt to predicttemperature gradients in asphalt pavements are either steady-state or one–dimensional transient approaches that fail to account for thermal interaction of parallel laid asphalt pavement lifts of varying grades and binders. The uncertainties associated with the pavement algorithms call for computationally fast tools that can accurately and reliably predict asphalt pavement temperatures at differentpavement depths and horizontal locations based on local ambient environmental conditions.

Low ambient temperatures and wind speeds create adverse conditions for hot-mix asphalt HMA paving. A HMA mix must be compacted to a specific range of air voids to control the density to achieve optimum mechanical properties. The time required for hot-mix asphalt to reach the proper compaction temperature decreases with an increased cooling rate (Chang *et al.,* 2009). In general, the minimum temperature allowed for compaction is set at 80^{o}C for the dense- graded mixture, AASHTO 2000.

This temperature is referred to as cessation temperature Roberts *et al.,* 1996; Foster 1970. As the mix cools, the asphalt binder becomes stiff to prevent further reduction in air voids regardless of the applied compaction efforts. Below the minimum compaction temperature, the gain of mix density with application of additional compaction efforts becomes difficult. Any additional rolling may result in fracture of aggregate and a decrease in density. Inadequate compaction of HMA would decrease the fatigue life, reduce strength and stability, increase moisture-related damage, and hence affect long- term pavement performance.

1.2Statement of Problem

Below the minimum compaction temperature for asphalt pavement, the gain of mix-density with application of additional compaction efforts becomes difficult. At such temperature, any additional rolling may result in fracture of aggregate and a decrease in density. Inadequate compaction of HMA would decrease the fatigue life, reduce strength and stability, increase permanent deformation, accelerate oxidation or aging, increase moisture related damage, and hence affect long term pavement performance.

Various methods for determining available compaction time have been developed using finite difference models Timm *et al.*, 2001; Tanet *et al.,* 1997; Tegeler and Dempsey 1973, Dickson and Corlew 1970. However, these methods use assumptions and typical values that may be inappropriate to predict the cooling rate for use in a tropical region like Nigeria. Hence the need to develop a model to adequately predict the appropriate time for asphalt laying in Nigeria

1.3Aim and Objectives

1.3.1 Aim

The aim of this research is to verify the appropriate time for asphalt pavement laying in Kaduna state. The time here refers to both the time for laying and temperature at the time of laying.

1.3.2 Objectives

The objectives of this study are as follows;

i. To investigate how the cooling time of HMA is affected by thickness, temperature and air voids.

ii. To investigate the value of the Traffic opening Time (TOT) at which the temperature variation becomes negligible and the paved road is open to traffic.

iii. To propose a model for local asphalt pavement laying in Kaduna state and its environs.

1.4 Justification of Study

The outcome of this study will serve as a guide to the government, consultant and contractors in Kaduna metropolis in predicting appropriate laying temperatures for road works as well as traffic opening time. Which will immensely help in solving road deteriorating problems associated with inadequate laying temperature, compaction and premature loading of newly constructed road pavements, this will in turn prolong the life span of the roadway pavement and delay the time when heavy investment is to me made on the road for either rehabilitation or reconstruction, thus, cost savings.

1.5 Scope and Limitation of Study

The scope of this study is to develop a model that verifies time and temperature of laying asphalt concrete in Kaduna State and its environs to predict appropriate asphalt laying temperature in terms of lift thickness, air temperature and percent air voids.

CHAPTER TWO

LITERATURE REVIEW

2.1 Introduction

A literature review was conducted to locate previous work done in the field of modeling temperature distributions in asphalt pavementslayingas a function of thermal environmental conditions.

Pioneering research in the field of asphalt pavement laying temperatures was done by Barber (1957). Barber attempted to correlate pavement surface temperatures and temperatures at 3.5 inch depths with standard weather report information. The weather parameters used were wind speed, precipitation, air temperature, and solar radiation.The pavement was considered to be a semi- infinite mass in contact with air. Barber observed that pavement temperature fluctuations measured roughly followed a sine curve with a period of one day. The research and analyses showed that when solar radiation was included in the analyses with air temperature, the sine curve approximation provided reasonable estimates of asphalt surface temperatures. Straub, *et al*., (1968) studied asphalt pavements in the northern climate of New York. The study considered dense graded pavements at various depths. Measurements of climatic parameters on site were made that influenced pavement temperatures.

A computer model was developed to predict agreed pavement layingtemperatures based on air temperature and solar radiation. The study showed that surface temperature measurements must be made at the surface to achieve a good correlation with solar radiation received at the site. Straub stated that temperatures at various depths of an asphalt pavement are independent of the thickness of the asphalt pavement. Results of the study also indicated that solar radiation had a greater effect on pavement surface temperatures than air temperatures, and that snow acted as an insulator while on pavement. The numerical model used by Straub subdivided the asphalt pavement in a column of nodes with a cross sectional area of one squaremetre. The nodes were set at the same depths as the thermocouples had been in the actual test section. Simulation of the model was initiated with guessed temperatures at each node. Energy balance equations for conduction, convection, and radiation were then applied to each node for the initial time step as applicable and a new set of temperatures was calculated for each node. At the next increment of time, the newly determined set of temperatures was used to determine pavement temperatures for the ensuing time step. This iterative process was continued over the time span of study, usually 24 hours. This model was found to provide a good correlation between measured temperatures and those predicted. The sensitivity of the model to the initial conditions used was checked for various depths. It was found that the predicted maximum surface temperatures are not sensitive to theinitial input values, but as depth increases the input temperature becomes more critical to the accuracy of the prediction.

Demsey and Thompson (1970) used an approach similar to that of Straub et al., to create a model to evaluate frost action in multilayered pavementslaying. The inputs required for the model included the climatic properties of short-wave and long-wave back radiation, convection, and air temperature. Inputs of thermal and material properties were unit weight, moisture content, material classification, thermal conductivity, and heat capacity of the pavement material. Temperatures predicted by the model were compared with those measured during the AASHTO road test and laboratory tests. Some of the input values, such as short-wave radiation, long-wave back radiation, and the convection coefficient, were not measured directly, but were estimated using empirical correlations based on previous research by Geiger (1959), and Vehren (1953).Air temperature values were not available at short enough time increments for use in the model, so intermediate values were approximated by a sine curve through the daily maximum and minimum air temperatures. Up into the late 1960s, pavement layingtemperature studies had only been conducted in the northern and central latitudes of the U.S.

Rumney and Jimenez (1971) filled the gap by conducting a study of pavement temperatures. In this hot desert climate, maximum pavement laying temperatures are the main concern to pavement engineers. Researchers were looking for a practical tool for predicting maximum surface temperatures. The study collected pavement laying temperatures at various depths, as well as the corresponding surface temperature and rate of incident solar radiation. From this data a set of correlations weredeveloped that predicted pavement laying temperatures for a given set of air temperatures and solar radiation intensities. Sets of curves were developed for pavement laying temperatures at 2- and 4- inch depths. Pavement layingtemperatures are of concern for pavement engineers in many climates worldwide. In South Africa, the primary consideration was the maximum pavement temperature in the upper levels of a pavement. Williamson (1972) developed a model by adapting a FORTRAN IV model developed by Schenk, Jr. (1963).

This model used finite-difference techniques to predict temperatures at various depths over a short period of time, usually a day. Inputs for the model included climatic parameters as well as the thermal properties of the pavement. Series of sensitivity analyses were performed investigating the impact of radiativeabsorption coefficient, surface emissive power, convection coefficient, thermal conductivity of the pavement material, pavement density, and possible errors in the measurements of incident radiation, initial temperature boundary conditions, and air temperature. Results of the analyses indicated that while variations in the surface absorption coefficient had large effects on temperatures, variations in other items, such as, emissive power, convection coefficient, and thermal conductivity had more marginal effects on temperature.

In addition, the model was validated using case studies. Data for model validation was collected from 8- inch thick asphalt pavement and Portland cement concrete pavement sections in Pretoria. Actual temperatures and predicted temperatures were plotted versus time for various 24-hour periods. The results showed a good correlation between predicted and measured temperatures. However, neither precipitation nor humidity effects have been considered in the model.

Christison and Anderson (1972) investigated the response of asphalt pavements to low temperature climatic environments. A computer model was developed that used a numerical finite difference method to predict the thermal regime in pavement systems. Christison andAnderson used the one dimensional, transient approach in a homogeneous pavement solving resulting energy balance equations with an implicit scheme. Input variables were the meteorological data, such as the air temperature, solar radiation, cloud cover and wind velocity, structural and physical properties including geometry of asphalt pavements, and thermo-physical properties of pavement materials. Comparisons between predicted temperatures throughout the asphalt pavement and measured temperatures showed excellent agreement at the Alberta, Canada, test site.

In Norway, Noss (1973) studied pavement temperatures related to frost penetration in subgrades. Using weather and pavement temperature data collected at the Vormsund Test Road, multivariate regression analyses were performed to predict the difference between air temperature and pavement temperature during cold winters. This boundary condition could then be used to calculate frost depth. The parameters included in regression analyses were 30-year mean air temperature (normal air temperature), relative difference between the normal air temperature and the recorded temperature, precipitation, wind velocity, cloud cover, relative humidity, and absorbed global radiation at the surface. Regression coefficients were calculated for various months.

Southgate and Deen (1969) developed a method of adjusting pavement deflection measurements to a reference mean pavement temperature using a five-day air temperature history. A linear relationship was found between pavement temperatures at a given depth and the sum of the surface temperature and the five-day mean air temperature history. The method was developed using data from Maryland. A model validation was performed using data from Arizona and New York. The study showed that the model worked equally well for additional data sets from radically different climates.

To better quantify the energy transfer experienced by an asphalt pavement during in-place recycling, a laboratory study was conducted by Highter and Wall (1984) to determine thermal properties of various asphalt mixes. During a typical recycling process, an external heat source is applied to the pavement and then approximately the top one inch is scarified and recompacted, and sometimes overlaid with additional pavement. A study by Carmichael *et al*., (1977) attempted to model the temperatures achieved during such a recycling operation. The study found that even a temperature as high as 540^{o}C applied for 30 seconds only changed the temperature to a depth of 16 mm (0.64 in). This simulation was based on estimated values for thermal conductivity, density and specific heat of asphalt pavement.

Highter and Wall study endeavored to measure these critical parameters as well as diffusivity of various pavement materials. The research showed that limestone mixes behaved differently from those prepared with lightweight aggregate. For the limestone mixes, thermal conductivity varied as much as 20 percent as the asphalt content was varied between 3.5 percent and 6.5 percent. Little variation of thermal conductivity in the lightweight aggregate mix was observed with comparable changes in asphalt content.

Specific heat was approximately 60 percent greater in the lightweight mix than in the limestone mix. Researchers noted that this primarily was due to the difference in unit weights. A significant difference was noted in the diffusivity of the limestone surface course and the limestone base course, apparently due to gradation and aggregate size. Thomson, Dempsey *et al.*, (1987) developed climatic a database for the State of Illinois. This database was derived from weather station records in 23 locations in and near the state. Maps were developed showing areas of equal percent sunshine and wind speed. A table of average weekly high and low air temperatures also was produced. Using this new database, combined with a heat transfer model developed years earlier, several new applications could be made. In one application, pavement temperatures were computed with the heat transfer model and climatic data.

A regression analysis was run to establish a relationship between pavementtemperatures and Mean Monthly Air Temperatures (MMAT). This information could then be used for selection of the proper asphalt cement modulus value to be used in pavement design. Wolfe *et al.,* (1987) suggested a “simple” predictive method based on heat transfer equations to determine the cooling rate of a freshly laid asphaltic mat under a given set of environmental conditions. The method was developed as a decision aid to help pavement engineers decide whether to proceed with construction on a daily basis. Research indicated that the cooling rates of mats of sufficient thickness can be slow enough to permit satisfactory compaction even under rather adverse weather conditions and temperatures.

Huber, *et al.,* (1989) adapted a computer program originally created to predict long term permafrost thawing over a period of years. The focus of the research was on studies to develop methods for the prediction of pavement thaw onset, so that the time allowable for use of heavy trucks for logging during the winter could be maximized. In the original program by Hildebrande and Haas (1983), the energy balance at the surface was modeled using so-called N-factors rather than surface temperatures. N-factors represent the ratio of daily mean air temperatures to daily mean pavement temperatures. Researchers were interested in predictions over shorter time spans, hours rather than months, so they adapted the program for input of pavement temperatures. Pavement temperature data were scarce for Saskachewan, so local data were used from a 1975 study by the Saskachewan Highway and Transportation Department. Temperatures were available on a bi- hourly basis. Thermal properties for the various layers were assigned for the thawed and frozen conditions. These values were modified until a good correlation with measured pavement and subsoil temperatures was obtained with the model. Hsieh *et al*., (1989) developed computer models for predicting temperatures in concrete pavements and rain water infiltration into soil and subgrades due to weather changes. The models use an implicit finite difference scheme that employs spatial factorization to implement the solution as an alternating-direction implicit sequence. The model utilizes a series of TMY (Typical Meteorological Year) weather databases pertaining to various climate conditions. Thesignificance of the study is that a three-dimensional numerical modeling approach was used coupled with moisture diffusion into pavements. An experimental validation of the model was attempted using data from sunny and cloud covered days in Miami and Orlando. Another area of interest to researchers was the cooling of fresh hot mix asphalt pavement. This area was of particular importance to Saskatchewan Highways and Transportation because of the short paving season in the northern latitudes. Engineers frequently struggle with whether or not to allow paving to proceed, either due to marginal weather conditions or the specified cut-off date has passed. In an attempt to aid the engineers in making these decisions, White, *et al.,* (1990) developed a method to predict available compaction times that could be used in the field. Using the TEMPR2 computer program developed by Jordan and Thomas (1976) and through dimensionless analysis, the sensitivity of heat flow variables weretested. The study showed that the most significant factors were initial mix temperature and lift thickness. Other importantvariables were wind speed, thermal conductivities, thickness of the existing pavement, ambient temperature, and incident solar radiation. A series of charts were developed from which the allowable compaction time could be estimated for various combinations of the variables. Studies also have been conducted on rigid pavements. Choubane and Tia (1992) measured pavement temperatures at various depths in rigid pavements in Florida.

A quadratic equation was developed that is used to predict temperature profiles in

Portland cement concrete pavements. The advent of the Strategic Highway Research Program steered research in a slightly different direction. The performance-type specifications developed for asphalt cements required that a certain grade of asphalt binder perform over a given range of temperatures. For pavement engineers, knowing the upper and lower temperatures a pavement would be exposed became important. Solaimanian and Kennedy (1993) made an effort to develop a simple way for pavement engineers to determine these critical temperature extremes.

Solaimanian and Kennedy study indicated that the difference between maximum pavement surface temperatures and maximum air temperatures were a function of latitude. A parabolic equation was developed thatdescribes this relationship well. Using a known value of latitude, the maximum expected surface temperature could be approximated. The study also recommended using the lowest expected air temperature as the lowest expected pavement temperature for design. A third order polynomial corollary equation was suggested to predict pavement temperatures at various depths.

Another study to predict effective asphalt layer temperatures was conducted by Inge and Kim (1995), who developed a database approach for the estimation of asphalt concrete mid depth temperature. The method represents improvements over the AASHTO method for the temperature correction procedure for asphalt concrete deflections in that air temperatures for the previous five days are not needed allowing for quicker computations, and heating and cooling cycles of asphalt pavements are taken into account. The research also studied an alternative temperature prediction model known as the BELLS equation to validate temperature prediction at on-third asphalt depths. Voller *et al.,* (1998) developed a computer tool to predict the time-dependent thermal profile in an asphalt concrete lift during pavement construction. The key feature of the computer model was that the thermal predictions were directly related to the compaction characteristics of an asphalt lift using an asphalt thermal properties database. The goal of the research was to develop a tool that can be used to determine optimum strategies for paving operations, particularly in cold climates. The proposed thermal model uses the one-dimensional, transient heat diffusion equation and subdivides the asphalt pavement into two regions: an asphalt lift and a ground base at the interface of which a constant conduction heat flux is assumed. The numerical model imposes a radiative and convective heat exchange boundary condition at the pavement surface while the bottom surface in contact with the earth is treated as an insulated boundary. The model requires 24-hour weather data for input as well as input of the thermal properties of the pavement materials.

Lukanen*et al.*, (1998) suggested a probabilistic method for asphalt binder selection based on pavement temperatures. The Lukanen*et al.*, study developed an empirical prediction model based on simple regression analysis to relate the seven-day average high air temperature to the seven-day average high pavement temperature. The analyses used data from SHRP obtained in Canada and the US as well as data from LTPP-SMP. Temperature prediction of the empirical model is then compared to existing prediction relationships including an asphalt pavement heat flow model.

Mohseni (1998) proposed revisions to the SHRP performance grading system for asphalt binder selection, specifically for low temperature applications. The study, based on data from Long Term Pavement Performance Study’s Seasonal Monitoring Program (LTPP-SMP), presents revised models for determining the low- and high-temperature component of Superpave performance-based binders. The study also compares existing models and resulting performance grades with the proposed approach. An interesting study was conducted by Bosscher*et al.,* (1998) on six test sections on US-53 in Trempealeau County, Wis., by using different performance-graded asphalt binders to validate the Superpave pavement temperature algorithm and the binder specification limits. The analysis was focused on development of a statistical model for estimation of low and high pavement temperatures from meteorological data. The model was then compared to the Superpave recommended model and to the more recent model recommended by the LTPP program. Although the temperature data analyses indicated a strong agreement between the new statistical model and LTPP model for the estimation of low pavement design temperatures, LTPP and Superpave models both underestimated the high pavement design temperature at air temperatures higher than 30C. The temperature data analyses also showed that there are significant differences between the standard deviation and of air temperatures and the standard deviation of pavement temperatures. The study raised questions about the accuracy of the reliability estimates used in the current Superpave recommendations.

2.2 Ambient Temperature

There are three basic steps to checking temperatures to assure a good and successful paving and patching projects. The first step is monitoring the ambient temperature, looking up to the expected high and low temperatures for the day of paving, as well as monitoring ambient temperatures during the work hours, is important to starting and maintaining a successful finished paving project. The normal requirement is that the ambient temperature should be 50^{o}F and rising on a paving or patching project.

It is also important to find out the projected wind velocity for the day of paving. When there is wind, the temperature of the hot mix asphalt pavement cools faster than normal. The higher the wind velocity, the quicker the hot mix asphalt will cool. It is also important to note than any precipitation can reduce temperature of the hot mix asphalt, which will hamper the effort to achieve required compaction.

2.3Base Compaction

Air temperature is a factor in cooling hot mix asphalt pavement, but the base or ground temperature is more critical. The second step is base or ground temperature. Monitoring the base (ground or existing pavement) temperatures can be accomplished with infrared thermometer to assure the base temperature is 50^{o}F.

2.4 HMA Asphalt

The final step is checking the temperature of hot mix asphalt prior to installing it on the base. This should be accomplished by taking temperature of the pavement in the haul truck, at the front of the lay down machine, and behind the screed (after lay down machine has passed).

Also, wind will cool the hot mix asphalt very rapidly after it has been placed on the base so caution should be taken when paving on windy days and break down rolling will need to be adjusted for the effects of wind velocity.

If the base or ambient temperature is not going to reach the minimum temperature requirement, there is chance of having a failing end product where the pavement will ravel and fall apart.

Similarly, cold-delivered asphalt mix will also cause the pavement to ravel and fall apart. Should the pavement cool too quickly and drop below 220^{o}F prior to the initial or break down rolling, failure will occur because the hot mix asphalt have set and required compaction (95% laboratory control) cannot be achieved.

Pavement temperature is very important to the roller operators to let them know when they need to do initial or break down rolling. Monitoring temperature and velocity are very important when installing new surface or when patching (especially skin or surface patching) on an existing paved surface.

The University of Minnesota and Minnesota Department of Transportation have developed a program named "Cool Pave" that can be used to determine the amount of time a contractor has to accomplish the break down rolling and achieve the required density.

2.5 Asphalt Concrete

Asphalt concrete (commonly called asphalt, blacktop or pavement in North America, and tarmac in Great Britain and Ireland) is a composite material commonly used to surface roads. Parking lots, and airports. It consist of mineral aggregates bound together with asphalt, laid in layers, and compacted. The process was refined and enhanced by Belgian inventor and U.S immigrant Edward de Smedt. It I increasingly being used as the core of embankment dams.

The term asphalt (or asphaltic concrete), bituminous asphalt concrete, and bituminous mixture are typically used only in engineering and construction documents, which define concrete as any composite material composed of mineral aggregate adhered with a binder. The abbreviation "AC" is sometimes used instead of asphalt concrete but can also denote asphalt content or asphalt cement, referring to the liquid asphalt portion of the composite material.

Mixing of asphalt and aggregate id accomplished in one of the several ways:

2.5.1 Hot mix asphalt concrete: Hot mix asphalt concrete (commonly abbreviated as HMAC or HMA) is produced by heating the asphalt binder to decrease its viscosity, and drying the aggregate o remove moisture from it prior to mixing. Mixing is generally performed with the aggregate at about 300^{o}F (150^{o}C) for virgin asphalt and 330^{o}F (166^{o}C) for polymer modified asphalt, and the asphalt cement as 200^{o}F (95^{o}C). Paving and compaction must be performed while the asphalt is sufficiently hot. In many countries, paving is restricted to summer months because in winter the compacted base will cool the asphalt too much before it is able to be packed to the required density. HMAC is the form of asphalt concrete commonly used on high traffic pavements such as those on major highways, race tracks and airfields. It is also used as an environmental liner for landfills, reservoirs, fish hatchery ponds. Superpave short for "superior performing asphalt pavement" is a pavement system designed to provide longer lasting roadways. Key components of the system are the careful selection of binders and aggregates, volumetric proportioning of ingredients, and evaluation of the finished product.

2.5.2Warm mix asphalt concrete: Warm mix asphalt concrete(commonly abbreviated as WMA) is produced by adding either zeolites, waxes, asphalt emulsion, or sometimes even water to the asphalt binder prior to mixing. This allows significantly lower mixing and laying temperature and results in lower consumption of fossil fuels, thus releasing less carbon dioxide, aerosols and vapors. Not only are working conditions improved, but the lower laying-temperature also leads to more rapid availability of the surface for use, which is important for construction sites with critical time schedules. The usage of these additive in hot mix asphalt may afford easier compaction and allow cold weather paving for longer hauls. Use of warm asphalt mix is rapidly expanding.

2.5.3Cold mix asphalt concrete: Cold mix asphalt concrete is produced by emulsifying the asphalt in water with essentially soap prior to mixing with aggregates. While in it emulsified state the asphalt is less viscous and the mixture is easy to work and compact. The emulsion will break after enough water evaporates and the cold will, ideally, take on the properties of cold HMAC. Cold mix is commonly used as a patching material and on less trafficked service roads.

2.5.4Cut-back asphalt concrete: Cut-back asphalt concrete is produced by dissolving the binder in kerosene or another lighter product of petroleum prior to mixing with the aggregate. While in its dissolved state the asphalt is less viscous and the mix is easy to work and compact, after the mix is laid down the lighter fraction evaporates. Because of concerns with pollution from the volatile organic compounds in the lighter fraction, cutback asphalt has been largely replaced with asphalt emulsion.

2.5.5Mastic asphalt concrete: Mastic asphalt concrete or sheet asphalt is produced by heating hard blown grade bitumen (oxidation) in green cooker (mixer) until it becomes a viscous liquid after which the aggregate mix is then added.

The bitumen aggregate mixture is cooked (matured) for about 6-8 hours and once it is ready the mastic asphalt mixer is transported to site where experienced layer empty the mixer and either machine or hand lay the mastic asphalt content on to the road. Mastic asphalt concrete is generally laid to a thickness of around 3/4 - 1 3/16 inches (20 – 30mm) for footpath and road applications and around 3/8 inches of an inch (10mm) for flooring or roofing applications.

In addition to the asphalt and aggregate, additives, such as polymer, and antistripping agents may be added to improve the properties of the final product.

2.5.6Natural asphalt: Natural asphalt can be produced from bituminous rock, found in some parts of the world, where porous sedimentary rock near the surface has been impregnated with upwelling of bitumen.

Asphalt concrete pavements especially those at airfields are sometimes called tarmac for historical reason, although they do not contain tar and are not constructed using macadam process. A variety of specialty asphalt concrete have been developed o meet specific needs, such as stone-matrix asphalt, which is designed to ensure a very strong wearing surface, or porous asphalt pavements, which are permeable and allow water to drain through the pavement for controlling storm water.

2.6 Performance Characteristics

Different types of asphalt pavements have different characteristics in terms of surface durability, tire wear, braking efficiency and roadway noise. In principle, the determination of the appropriate asphalt performance characteristics must take into account the volume of traffic in each vehicle category, and the performance requirements of the friction course. Asphalt concrete generate less roadway noise than Portland cement concrete surface, and is typically less noisy than chip seal surfaces (John, 1973).

Because tire noise is generates through the conversion of kinetic energy to sound waves, more noise is produced as the speed of the vehicle increases. The notion that highway design might take into account acoustical engineering considerations, including the selection of the type of surface paving, arose in early 1970s (Hogan, 1973).

2.7 Asphalt Concrete Degradation and Restoration

Asphalt deterioration can include crocodile cracking, potshots, upheaval, raveling, bleeding, rutting, shoving, stripping, and grade depressions. In cold climates, frost heaves can crack asphalt even in one winter. Filling the cracks with bitumen is a temporary fix, but only proper compaction and drainage can slow this process.

Factors that cause asphalt concrete to deterioration over time mostly fall into one of the three categories: construction quality, environmental considerations, and traffic loads. Often, damage result from combinations of factors in all three categories.

Construction quality is critical to pavement performance. This includes the construction of utility trenches and appurtenances that are placed in the pavement after construction. Lack of compaction in the surface of the asphalt pavement, especially on the longitudinal joint can reduce the life of a pavement by 30 to 40%. Service trenches in pavement after construction have said to reduce the life of pavement by 50%, mainly due to lack of compaction in the trench, and also because of water intrusion through improperly sealed joints.

Environmental factors include heat and cold, the pressure of water in the subbase or subgrade soil underlying the pavement, and frost heaves.

High temperatures softens the asphalt binder, allowing heavy tire loads to deform the pavement into ruts. Paradoxically, high heat and strong sunlight also cause the asphalt to oxidize, becoming stiffer and less resilient, leading to crack formation. Cold temperatures can cause cracks as the asphalt contracts. Cold asphalt is also less resilient and more vulnerable to cracking.

Water trapped under the pavement softens the subbase and subgrade, making the road more vulnerable to traffic loads. Water under the road freezes and expands in cold weather, causing and enlarging cracks. In spring thaw, the ground thaws from the top down, so water is trapped between the pavement above and still-frozen soil underneath. This layer of saturated soil provides little support for the road above, leading to the formation of potholes. This is more of a problem for silty or clay soils than sandy or gravelly soils. Some jurisdiction pass frost laws to reduce the allowable weight of trucks during the spring thaw season and protect their roads.

Traffic damage mostly results mostly from trucks and buses. The damage a vehicle causes is proportional to axle load raised to the fourth power, so doubling the weight an axle carries actually causes 16 times as much damage. Wheel cause the road to flex slightly, resulting in a fatigue cracking, which often leads to crocodile cracking. Vehicle speed also play a role. Slowly moving vehicles stress the road over a longer period of time, increasing ruts, cracking, and corrugation in the asphalt pavement. Other causes of damages include heat damage from vehicle fires, or solvent action from chemical spills.

2.8 Prevention and Repair of Degradation

The life of a road can be prolonged through a good design, construction and maintenance practices. During design, engineers measure the traffic on the road, paying special attention to the number and types of trucks. They also evaluate subsoil to see how much load it can withstand. The pavement and subbase thicknesses are designed to withstand wheel loads.

Sometimes geogrids are used to reinforce the subbase and further strengthen the roads. Drainage, including ditches, storm drains and underdrains are used to remove water from the roadbeds, preventing it from weakening the subbase and subsoil.

Good maintenance practices enters on keeping water out of the pavement, subbase and subsoil. Maintaining and cleaning ditches and storm drain will extend the life of the road at low cost. Sealing small cracks with bituminous crack sealer prevents water from enlarging cracks through frost weathering, or percolating down to subbase and softening it.

For somewhat more distressed roads, a chip seal or similar surface treatment may be applied. As the number, width and length of cracks increase, more intensive repairs are needed. In order of generally increasing expenses, these include thin asphalt overlays, multicourse overlays, grinding off the top and overlaying, in-place recycling, or full-depth reconstruction of roadway.

It is far less expensive to keep a road in good condition than it is to repair it once it has deteriorated. This is why some agencies place the priority on preventive maintenance of roads in a good condition, rather than reconstructing roads in poor condition. Poor roads are upgraded as resources and budget allow. In terms of lifetime cost and long term pavement conditions, this will result in better system performance. Agencies that concentrates on restoring their bad roads often find that by the time they have repaired them all, the roads that were in good condition have deteriorated. Some agencies use a pavement management system to help prioritize maintenance and repairs.

CHAPTER THREE

RESEARCH METHODOLOGY

3.1 Introduction

Various methods for determining available compaction time have been developed using finite difference models Timm *et al.,* 2001; Tanet *et al.,* 1997; Tegeler and Dempsey 1973; Dickson and Corlew1970. However, these methods use assumptions and typical values that may be inappropriate to predict the cooling rate for use in other regions.A model that is simple and applicable needs to be developed for the local paving projects. Previous researchers have studied the temperature distribution within a pavement for the purposes of binder selection and temperature segregation Diefenderfer et al. 2006; Mahoney *et al.,* 2000; Solaimanian and Kennedy 1993. Both dense-graded and porous mixtures will be used in this study. These mixtures were also selected to present two types likely to exhibit different cooling behaviors. The binder and the aggregate type in each of the graded asphalt will be determined. The binder content by weight of total mix will be determined by using the Marshall mix design method. Mixtures to be used in the laboratory will be the same mix as constructed in field testing sites.

After the mix design, specimens were compacted and tested for their susceptibility to temperature changes. Air voids and thicknesses of samples compacted cover the range of 6, 7, 8, 10, 15, and 20% and 5, 10, and 15 cm respectively being the typical asphalt lift. Digital thermometer was used to record and collect data every minute for an initial 30 min, every 5 min for another 1 h and then every 10 min for the rest of the time period. A 10-cm-diameter specimen was designed to contain HMA mix to simulate the field condition. The mix was heated to a mix laydown temperature of 140°C before taking temperature measurements.

3.2Field Tests

The test site is located at the Transport Technology Centre (TTC) of the Nigerian Institute of Transport Technology (NITT) where there is ongoing construction work and asphalt pavement of the test tract. Note that the mixes used in the field and in the laboratory were the same and the nominal maximum aggregate size will be 19 mm. Mixtures for this study were selected in order to represent two types of asphalt concrete that would show different cooling behaviors. These two types of asphalt concrete are dense- and open- and gap-graded mixes that varied in gradation. A dense-graded mix is generally easy to compact.

Open-graded mixes typically require an increase in compactive effort to obtain the desired level of density. In addition to data collected from the laboratory, and field temperature measurements were made at regular time intervals and documented for future model checks.

At the field site, Digital thermometers obtained from Federal Road Maintenance Agency (FERMA) were placed at the center of a paving lane to measure temperature changes at depths of 0, 2.5, 5, 7.5, and 10 cm below the surface. The thermometers were used to measure temperature changes during and after the paving operation.

3.3Effect of Air Voids

Asphalt mixes of various air voids was compacted to measure temperature changes. Air voids of 6, 7, 8, and 10% were chosen for a dense mix, and air voids of 15 and 20% for a

PA mix. The cooling process of the compacted is categorized into three stages which are rapid, transition and stable zones.

3.4Development of Cooling Model

A model that explains the cooling behavior of a HMA mix is needed to predict the time required to cool to 80°C. In the model building process, lift thickness, an air temperature and air void of compacted HMA was used as predictor variables in the multiple linear regression analysis. The air temperature was obtained from Nigerian College of Aviation Technology (NCAT), Institute for Agricultural Research (IAR). The selection of predictor variables was performed with the assistance of SPSS statistical software.

Algorithms including stepwise regression, forward selection, and backward elimination were employed to develop a quantitative model. The diagnostics for the preliminary models was applied to check if the coefficient of each variable was appropriate in accordance with the cooling behavior of asphalt mixtures. In addition, scatter plots and residual plots were examined to determine relationships and strengths of the fitness in the regression models. Thickness, ambient temperature, and air voids were selected as independent variables in the proposed model. These independent variables were easily acquired in the field and from the local weather report. The model is in the following general form:

$$Y\ T80 = \mathrm{A}XD + \mathrm{B}XA + \mathrm{C}XV + \mathrm{D}$$

Where $Y\ T80$ = time required to cool to 80°C min; XD = lift thickness cm; XA = air temperature °C; and XV = percent air voids%. A, B, C are the regression constants for lift thickness, air temperature and air voids respectively. D is the constant of the regression equation.

Thevalue is used to verify the variation in time required for a mix temperature to drop to 80°C which can be explained by the pavement thickness and the air temperature. The regression relation between the response and the predictor variables was evaluated using the *F*-test.

In developing the cooling model, the thermal properties of HMA are considered ashomogeneous without spatial variation, and the heat transfer is assumed to be one dimensional along the vertical direction. The cooling time is also assumed to be unaffected by changing moisture conditions. In general, the use of thermal properties that are not corrected for changing moisture conditions was considered to furnish good prediction results provided the moisture changes are small.

To verify the applicability of the cooling model, the times available for compaction obtained from the laboratory and the field site were compared with those to be calculated from the proposed model.

It is noteworthy that the statistical parameter of the model developed in this work wasapplicable only to the environmental conditions and materials used in this study.

CHAPTER FOUR

RESULTS AND DISCUSSION

4.1 Mechanical Properties

Fig. 4.1 shows the stability value and the indirect tensile strengthversus temperature. The following results were obtained and from which graphs were plotted.

Table4.1: Marshall Stability

STABILTY (kN)	TEMPERATURE (^oC)
39.5	25
24.0	40
14.5	50
8.5	60
8.0	70
6.5	80
5.5	90

Table 4.2: Indirect Tensile Strength

TENSILE (kN)	TEMPERATURE ^oC
10.0	25
4.0	40
1.0	50
0.7	60
0.5	70
0.4	80
0.3	90

Each data point represents an average of threesamples for a given temperature. The trend indicates that bothstability and indirect strength decrease with increasing temperature. There is a tenfold increase in indirect tensile strength as mix temperature drops from 90 to 10° C primarily due to an increasein binder viscosity.

Laboratory data show increase in asphalt viscosity as the temperature drops from 140 to

50° C. The stability value at 80° C is approximately 50% of that at 50° C, and the indirect tensile strength is approximately 35%

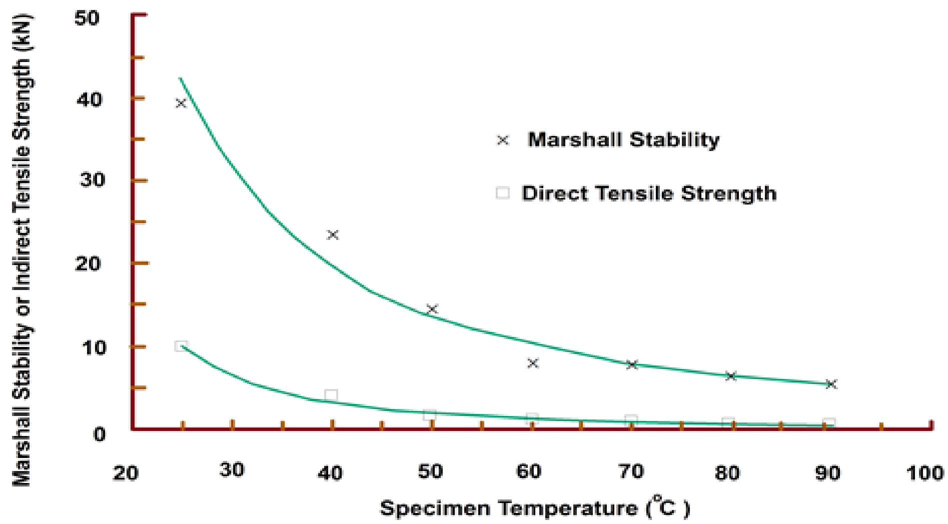

Figure: 4.1 Influence of temperature on Marshall Stability and indirect tensile strength

At relatively high temperatures, say higher than 80° C, an asphalt mixture acts more like a viscous material that is unable to provide adequate stability and strength to carry traffic loads. The resilient modulus values at 60 and 70°C are normalized with respect to the respective values measured at room temperature 50° C, as shown in Fig. 5. As expected, with increasing pavement temperature the mean modulus decreases. The resilient modulus testing at 60° C is about 43% of that at 50°C and significant reduced to 12% as the temperature is raised to 70° C. Thus, the mix temperature is recommended to be at least below 50° C before pavements are allowed to open traffic.

4.2 Effect of Low Temperature

The cooling data were recorded at two different air temperatures of 16 and 25° C. The temperature of 25 and 16° C is the average night temperature in summer and in winter, respectively, in Zaria. Fig. 6 shows the cooling curves for a lift thickness of 15 cm at various depths. The temperature measurement was terminated at the time when the bottom temperature reached 50°C.The cooling rate for temperature 25° C is lower than that for a temperature of 16° C. The trend is similar for temperatures measured at various depths. During low temperature time construction, convection plays a dominant role in transferring heat between pavement surface and air whereas conduction through the solid skeleton plays a separate role in transferring heat within the pavement system. It is apparent that radiation is also a major factor during night construction. As discussed previously, the heat loss is related to the temperature difference between HMA and ambient or base conditions. The temperature difference increases with the increasing rate of heat flow by convection or conduction to its surroundings, especially when air temperature or base temperature is lower than 16° C.

Table 4.3: Table of ModulusVersus Temperature

MODULUS (N/M^2)	TEMPERATURE (^oC)
1.00	50
0.51	60
0.11	70

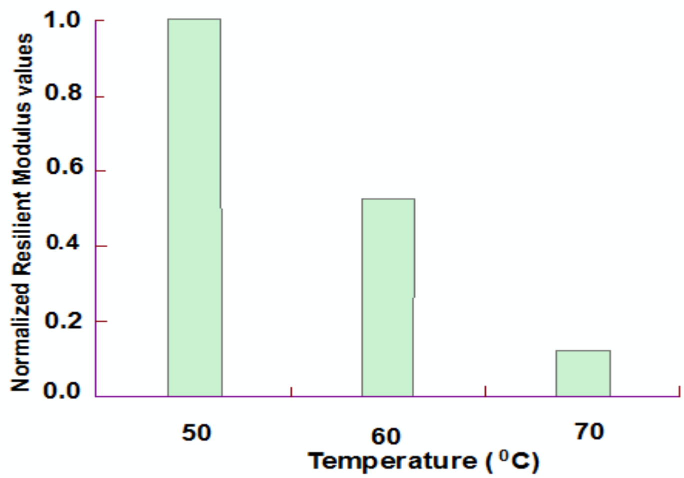

Figure:4.2 Normalized value of resilient modulus at different temperatures

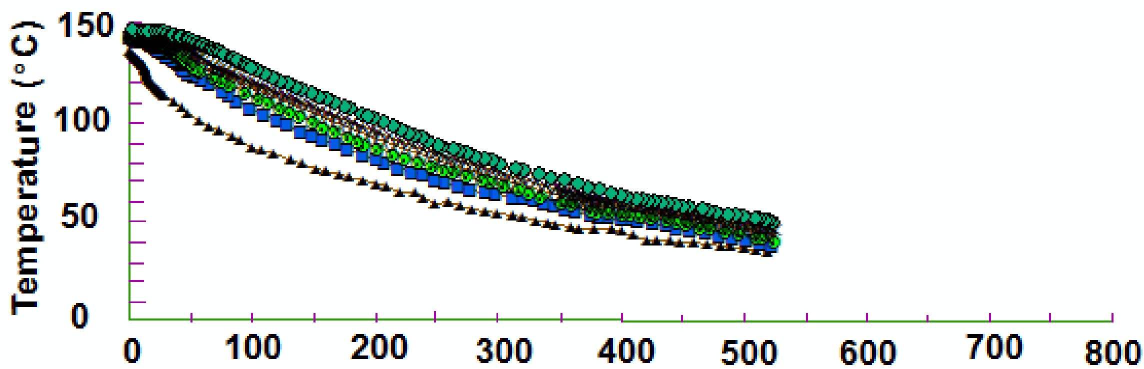

Figure 4.3 and 4.4 Cooling curves for 15-cm-thick specimen at night temperature of 16° C

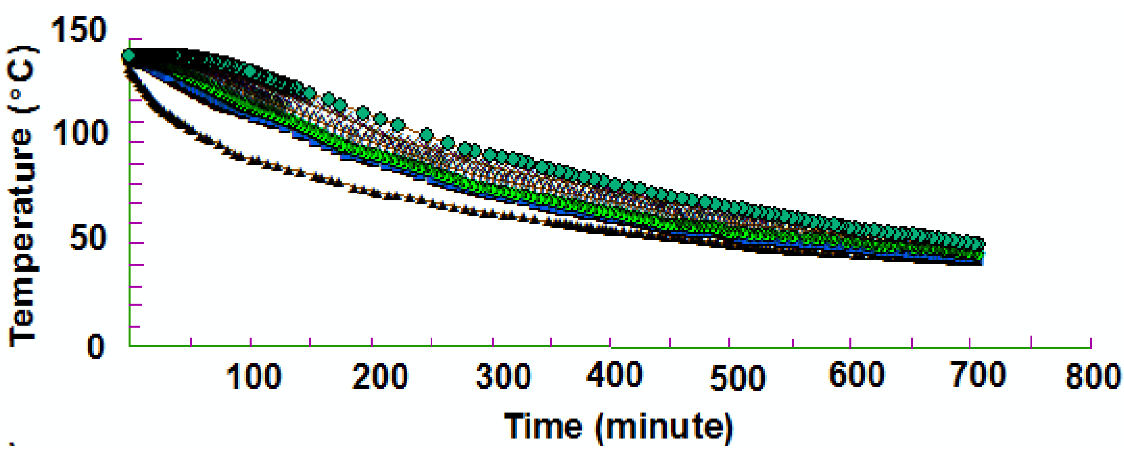

Figure 4.3 and 4.4 Cooling curves for 15-cm-thick specimen at night temperature of 25° C

The cooling time for the temperature of a dense-graded mix to drop to 80 and 50° C is given in Table 4.4 Test results indicate thatan increase in the ambient temperature increases the cooling time. The cooling time for a 10 cm lift from 140 to 80° C is 78 min at a temperature of 16° C as compared to 100 min at a temperature of 25° C. For a mix temperature of 140° C and a lift thickness of 5 cm, it takes 189 min for the mix to cool to 50° C when the temperature is 16° C. The change in temperature from 16 to 25° C causes an increase in cooling time by a factor of 1.2–2.0 depending on the lift thickness. Although 50°C is generally considered as the mat temperature for open traffic, it is sometimes impossible to close traffic for more than 2h because of worsening congestion.

Table 4.4: Cooling Times for Temperature Drop to 80 and 50° C

Night Temperature °C	Lift thickness cm	Cooling Times min	
		Cools to 80 °C	Cools to 50 °C
	5	71	189
16	10	78	235
	15	135	347
	5	93	218
25	10	160	351
	15	174	512

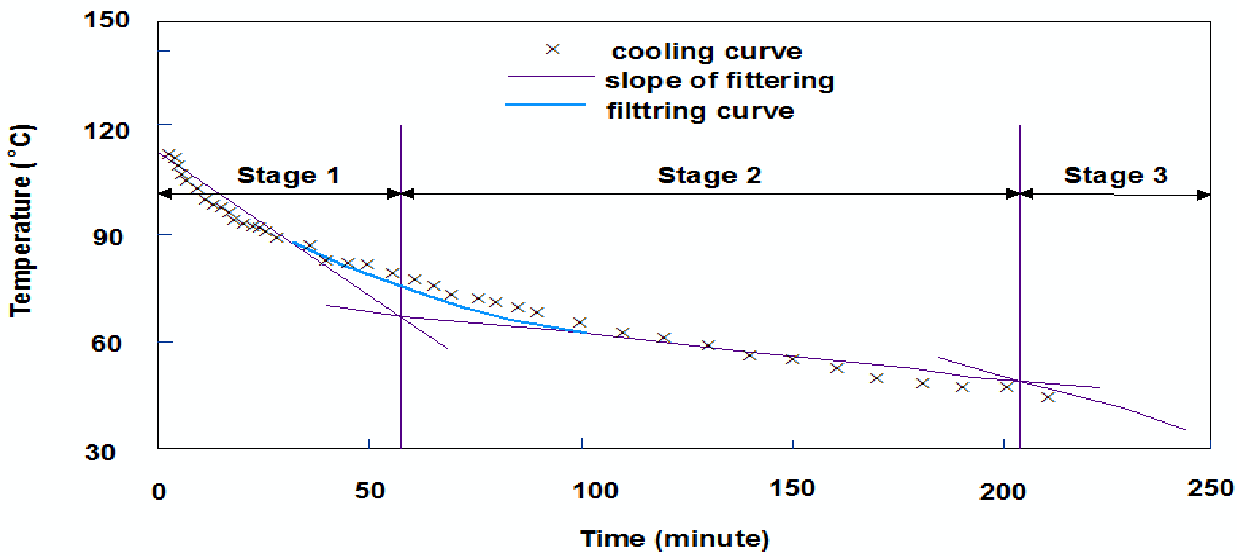

Figure 4.5:Illustration of three stages in pavement cooling process

During the paving operation on a highway, compaction equipment needs to be used more effectively.

4.3 Effect of Air Voids

Asphalt mixes of various air voids were compacted to measure temperature changes. Air voids of 6, 7, 8, and 10% were chosenfor a dense mix, and air voids of 15 and 20% for a PA mix. The cooling process of the compacted mix can be categorized into three stages as shown in Fig. 4.5. The cooling rate is defined as the temperature change per minute in a HMA mat, and can be represented by the slope of the cooling curve. The separation of each stage is based on a three-order polynomial regression in which the rate of slope change is used as a criterion to differentiate the stage. The change of the cooling rate can be determined by the second differential of the regression line. In the first stage, the cooling rate is high because heat transfer is proportional to the temperature difference between ambient conditions and the HMA mixture. A large portion of the heat in the surface layer is lost to the air by convection. In the second stage, the cooling rate is slower because heat energy is gradually transferred through heat conduction from bottom flowing into the upper portion of pavements in the direction of decreasing temperature. Cooling of the mix near the base is not as rapid as near the surface because the air temperature is relatively high. In the third stage, the mix approaches the equilibrium condition of heat transfer since the mix temperature is close to air and base temperatures. Accordingly, these three stages can be classified into rapid, transition, and stable zones.

Figure 4.6 shows the cooling curves measured on the surface of dense-graded mixes with various air voids. These temperature data are separated into three stages according to the classification of the cooling rate as shown in Fig. 4.5.

The effects of air voids onthe cooling rate are analyzed. The procedure is based on a ranking by the size of thepooled samples rather than their nominal temperature values. The tests were conducted in making inferences about different treatment population means Neter et al. 1993.

The null hypothesis is that no temperature differences among samples of different air voids could be observed. At the 0.05 significance level, the test statistic cannot reject the null hypothesis. Limited differences were observed in temperature drop for dense-graded mixtures with air voids up to 10%. It implies that the thermal conductivity of a well compacted dense mixcould be controlled, thus reducing the effect of air voids on heat propagation. Similar observations are also reported by other researchers Mrawira and Luca (2006).

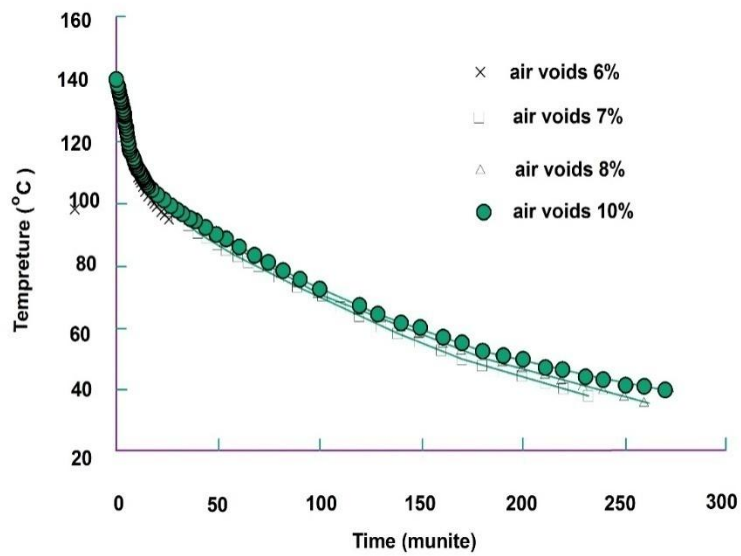

Figure 4.6: Cooling curves for dense-graded mixture at various percentair voids

The cooling curves for a dense-graded mix with 10% air voids and porous mixes with 15 and 20% air voids are demonstrated inFigure. 4.7. As shown in Figure4.5, the cooling process can be separated into three segments, i.e., 0–45, 45–190, and more than 190 min for Stages 1, 2, and 3, respectively. During the first stage, the slope of the cooling curve increases with increasing air voids when the mix temperature is above 80° C. The duration for temperature reduction from 140° C to a cessation temperature of 80° C is 67, 51, and 32 min for air voids of 10, 15, and 20%, respectively. According to the statistical test, the cooling rate with temperature drop to 80° C is significantly different for these three mixes with p value equal to 0.004 in the first stage. It implies that, for a porous asphalt pavement with air voids of more than 15%, immediate compaction is needed to obtain the required engineering properties while the mixture is in a workable condition. Special attention should be paid to low temperature construction to obtain the controlled air void within a reasonable time frame.

In the second and third stages, the temperature difference is found to be insignificant for samples of different air voids. As thethermal energy approaches equilibrium, the overall effect of convection on samples gradually reaches balanced. The heat transfer in the third stage is mainly due to the aggregate-to-aggregate conduction.

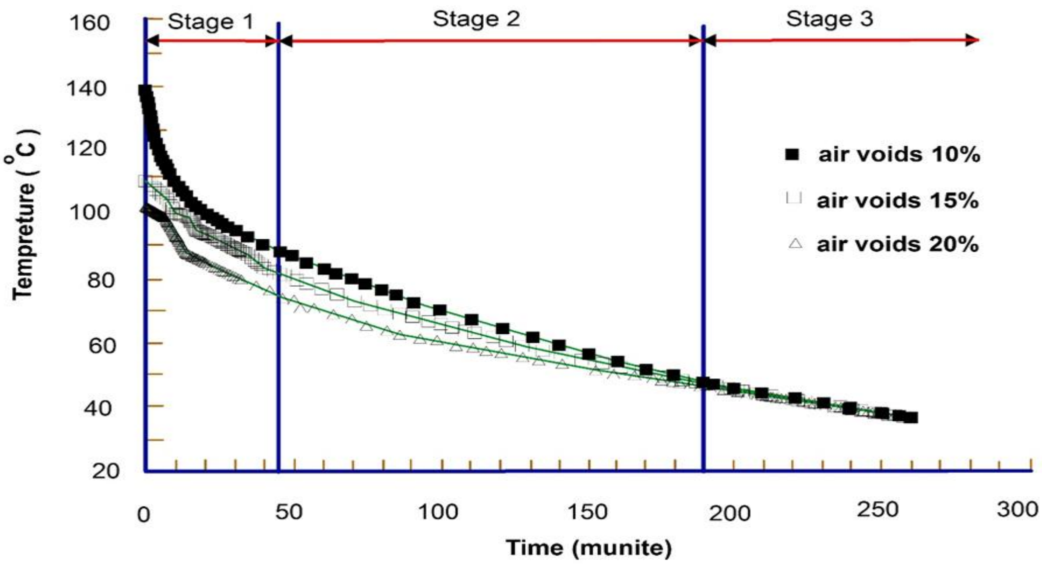

Figure 4.7: Comparison of cooling curves for mixes with various airvoids

Table 4.5 Cooling curves table for PA specimen 5cm thick

Depth(0cm)	Depth (2.5cm)	Depth (5cm)	Time (Min.)
0	0	0	0
93.6	109.8	129.0	40
79.8	93.6	109.8	80
67.2	78.0	90.0	120
58.8	69.0	74.4	160
49.8	58.8	68.0	200
40.8	48.0	53.4	240

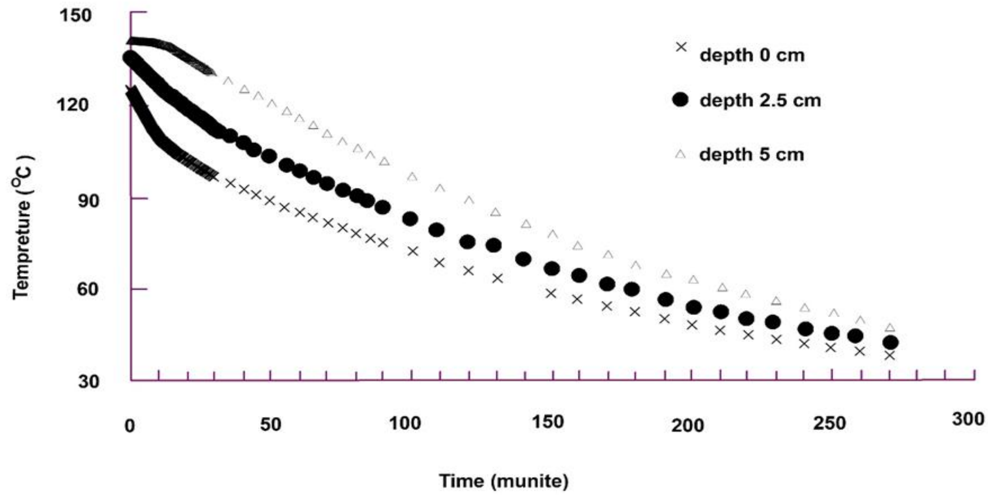

Figure4.8: Cooling curves for PA specimen 5 cm thick

4.4 Effect of Lift Thickness

Layers of mix equal to 5, 10 and 15 cm thick were prepared for measuring the cooling rate. Temperatures were measured at a2.5-cm interval depth below surface. The mix temperature and the cooling time were recorded from initial 140° C and stopped when the temperature reached 50° C. Fig.4.8 shows the cooling curve for a 5 cm thick at depths of 0, 2.5 and 5 cm. The cooling curve at 0-cm depth has a steeper slope for temperatures between 140 and 120° C, and the slope decreases as the cooling process continues. The relatively rapid cooling process on the pavement surface is due to heat convection. The rate of heat energy transferred to the surrounding air is directly related to the exposed area of the HMA mix and the temperature difference between HMA and air. The highest cooling rate occurs when the temperature difference reaches a maximum valuei.e., 140° C − 25° C = 115° C initially, then decreasing with time. The cooling rate at 2.5-cm depth is low as compared to that on the surface, i.e., 0-cm depth. The cooling curve at 5-cm depth shows an even lower cooling rate initially due to the interaction between HMA and base layer, then increasing after 20 minutes. Other specimens also show similar trends. Although convection and radiation are necessary components of a pavement cooling model, they are considered to have a minor effect on lift thickness. Conduction is the primary factor that affects the transfer of heat through a solid, andis the basis of thickness effect required for pavement cooling. The effect of conduction on lift thickness can be described by Fourier's law, which states that the heat flux in a given direction is proportional to the temperature gradient in the direction. It implies that the thicker the lift, the longer it takes for the heat of the lift to dissipate, and consequently, the longer is the period available to compact the lift.

Figure 4.9 shows the cooling rate in a logarithmic scale for the 5-cm-thick specimen. The cooling rate measured at three different depths is different. Initially the cooling rate measured at 5 cm depth is the lowest. A decrease in temperature from 140 to 80°Cresults in a 50% decrease in the cooling rate for the dense-graded mix and a decrease by over 90% for the PA mix. As the cooling process continues, the cooling rate becomes stable and maintains approximate 0.4^{o}C/ min. similar trends are also observed for specimen thicknesses of 10 and 15 cm.

Fig. 4.10 shows comparisons of cooling curves for samples with 5, 10, and 15 cm thickness measured at 2.5 cm depth for thedense-graded mix. A decreasing trend in the cooling rate is expected as the thermal conductivity of asphalt concrete decreaseswith decreasing temperature difference between air and mix. The time required for a sample to cool to 80 and 50° C increases withincreasing thickness.

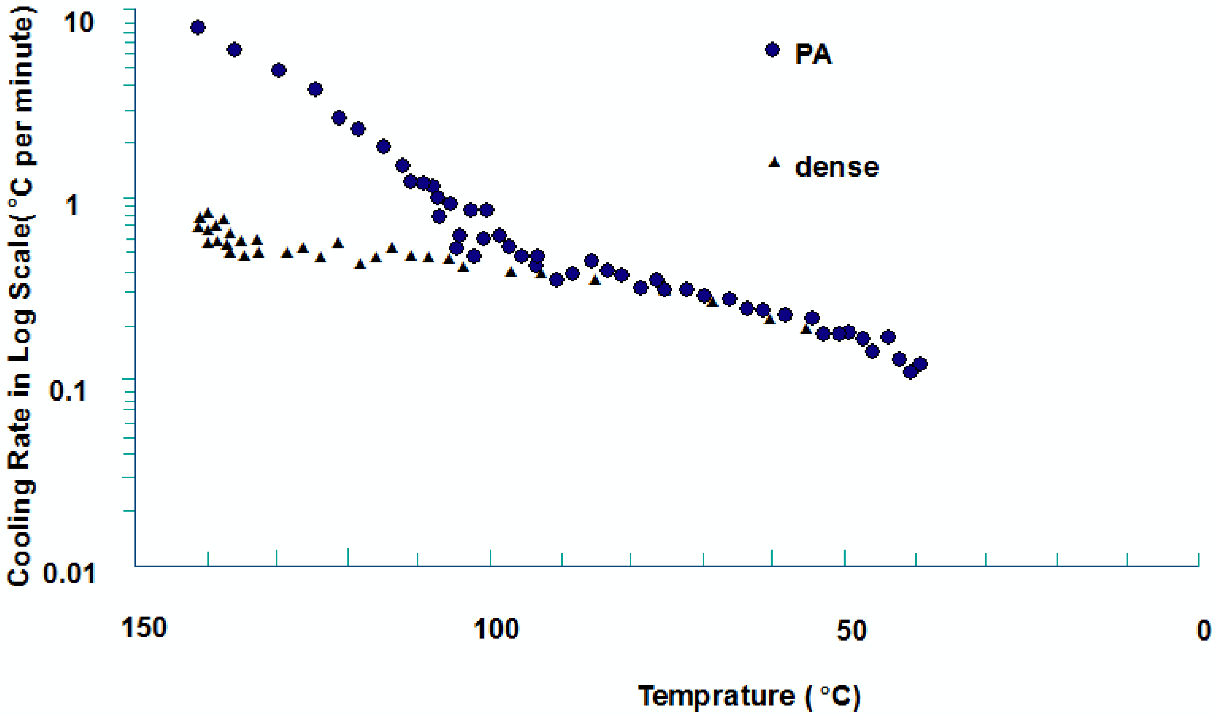

Figure 4.9: Cooling rate versus time for specimens 5 - cm thick at depth2.5 cm

As the thickness of the layer being placed increases, the time available for compaction also increases.

The time available to cool to 80 and 50° C at a specific depth for mixes of various thicknesses is shown in Fig.4.11. The initial HMA temperature was controlled at 140° C and the air temperature was kept at 25° C. The time for the specimens to cool to 50° C increases by two times as the thickness increases from 5 to 10 cm. The relation between the cooling time to 50 or 80° C and the thickness of the sample appears to be linearly related. The required time to cool to 80° C surface temperature at 0 cm depth is approximately 93, 160, and 174 min for 5, 10, and 15 cm thickness, respectively. During the low temperature time paving operation, the time to achieve effective compaction above the cessation temperature can be estimated using the data provided in Fig.4.11. For an increase in thickness from 10 to 15 cm, the gain of extra compaction time seems to be limited, increasing from 160 to 174 min.

An increase in pavement thickness that is greater than 10 cm slightly increases the available compaction time. This demonstrates that an increase in lift thickness to 15 cm may not be necessary to gain more time for compaction.

4.5 Comparison of Field and Laboratory Measurement

The comparison of test results between the field and the lab data. Both field and lab mixes were compacted to the same air void. During the compaction process of a paving project, the steel-wheeled roller was used for the breakdown position, and the pneumatic roller was used for intermediate position and the finish rolling. Because a HMA mix may stick to the tires during rolling, water was sprayed to prevent adhesion of the mixture to the rolls. The change in the pavement temperatures due to the spray water was measured.

Table 4.6 Cooling curves table at depth 2.5cm for specimens 5, 10, 15cm

5-cm thick	10-cm thick	(15-cm thick	Time(Min)
87.5	110.0	117.5	100
57.5	87.0	95.0	200
	65.0	75.0	300
	47.0	65.0	400
		52.0	500
		45.0	600

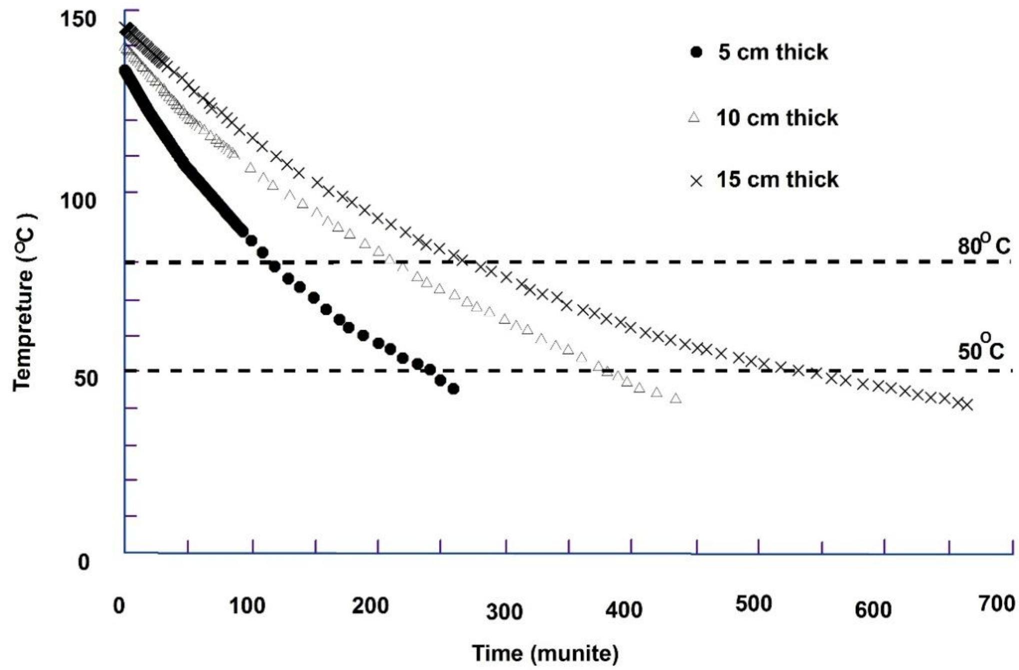

Figure:4.10.Cooling curves at depth 2.5 cm for specimens 5, 10, and 15 cm thick

Table 4.7: Temperature Cooling to 50 ^oC

Depth (0cm)	Depth (2.5 cm)	Depth (5cm)	Thickness (CM)
0	0	0	5
240	270	285	6
300	330	350	8
360	390	420	10
420	465	490	12
480	520	565	14

Table 4.8: Temperature Cooling to 80 ^oC

Depth (0cm)	Depth (2.5 cm)	Depth (5cm)	Thickness (CM)
0	0	0	5
110	135	150	6
125	155	210	8
150	200	230	10
150	230	265	12
160	260	300	14

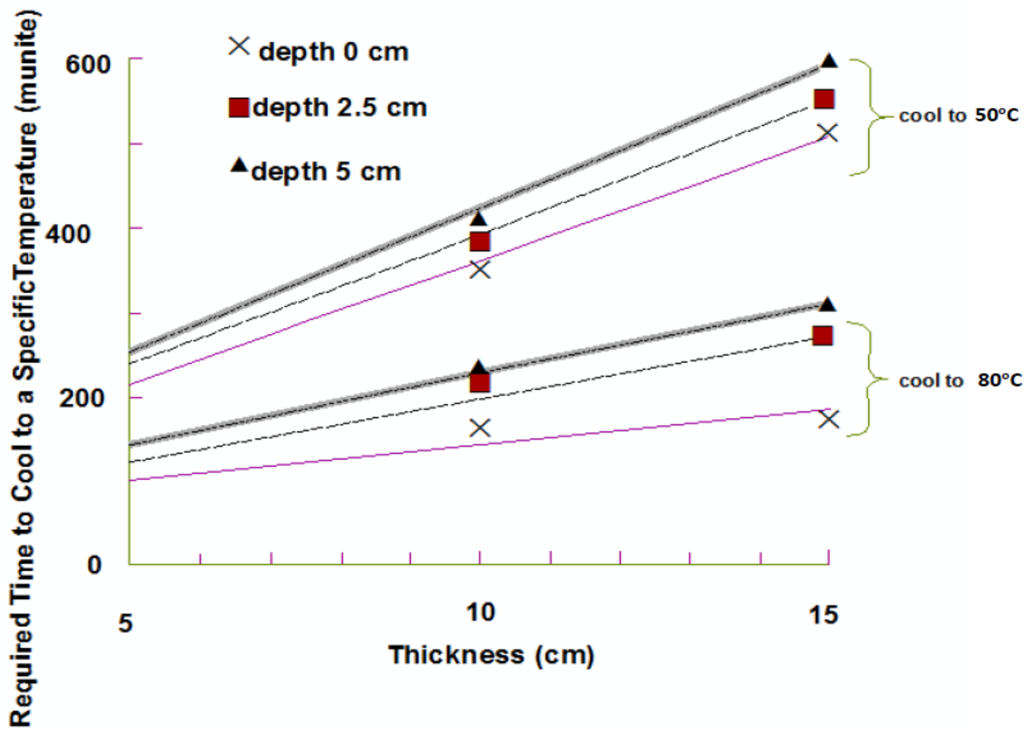

Figure: 4.11 Time required for specimen to cool to 80 and 50° C versusspecimen thickness

Using the same amount of water as used in thefield, cool water was also sprayed on the specimen in the laboratory to simulate the field rolling practice.

Figure 4.12shows the cooling curve with temperature changes measured on the pavement surface. Cooling rate for initial 20min at the test site is higher than that for lab specimens. Discrepanciesexisting in the beginning are unavoidable because of the influence of a large number of factors and their corresponding variations. As the cooling time is prolonged, the cooling curve measured in the field matches that in the laboratory. Fig. 4.13 indicates that the trends for both field and lab cooling processes are similar and

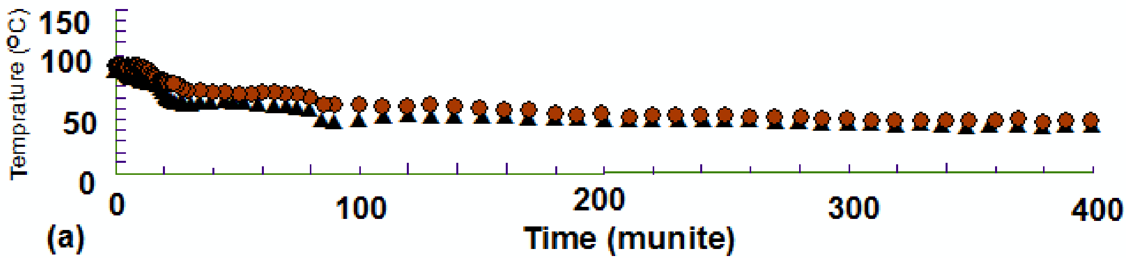

Figure: 4.12 Comparisons of lab-measured and field-measured temperatures: a at 0-cm depth

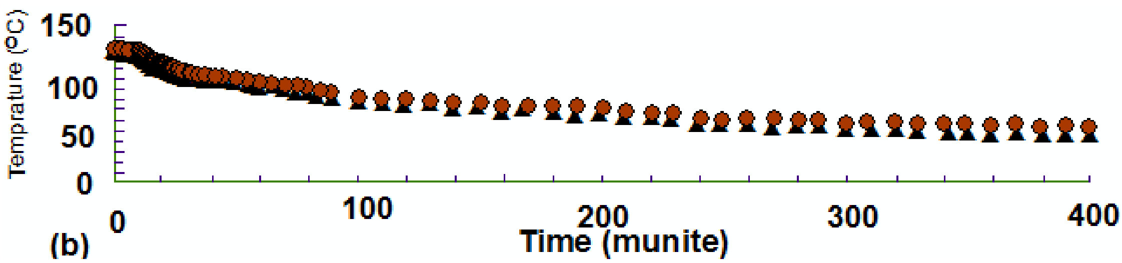

Figure: 4.13 Comparisons of lab-measured and field-measured temperatures: a at 2.5-cm depth

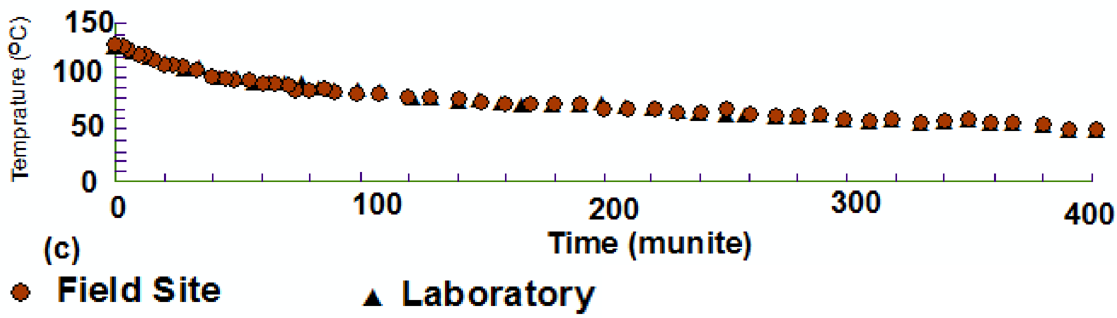

Figure: 4.14 Comparisons of lab-measured and field-measured temperatures: a at 5-cm depth

The difference in temperature is within single digit numbers. At a depth of 5 cm, the spray water would not influence the pavement temperature because water could be heated to the same temperature of HMA, as shown in Figure 4.14. Other field cases also show similar observations. Comparison of field and laboratory measurements at various depths indicates that within the top 15 cm of pavement, the differences are within 2.9 – 3.4° C.

4.6 Development of Cooling Model

A model that explains the cooling behavior of a HMA mix isneeded to predict the time required to cool to 80° C. In the model building process, lift thickness, air temperature, and air voids ofcompacted HMA were used as predictor variables in the multiplelinear regression analysis. The selection of predictor variableswas performed with the assistance of SPSS statistical software.

Algorithms including stepwise regression, forward selection, and backward elimination were employed to develop a quantitative model. The diagnostics for the preliminary models were applied to check if the coefficient of each variable is appropriate in accordance with the cooling behavior of asphalt mixtures. In addition, scatter plots and residual plots were examined to determinerelationships and strengths of the fitness in the regression models. Thickness, ambient temperature, and air voids were selected as independent variables in the proposed model. These independent variables can be easily acquired in the field and from the local weather report. The model is developed as follows:

$Y\,T80 = 6.76XD + 6.24XA + 0.02XV - 78.71 \quad R^2 = 0.84$

Where $Y\,T80$ = time required to cool to 80° C min.; XD = lift thickness cm;

XA = air temperature °C; and XV = percent air voids%.

The R^2 value indicates that 84% of variation in time required for a mix temperature dropped to 80°C can be explained by the pavement thickness and the air temperature. The regression relation between the response and the predictor variables were further evaluated using the *F*-test. The *F*-test with a *p* value of 0.00003 shows a strong relation between the cooling time and the variables selected in the model. Note that this model is only applicable to conditions tested in this study and the seasonal temperature effects can be taken into account by changing the air temperature.

The significance of this cooling model is that it is possible to predict available compaction time from easily acquired data such as lift thickness and ambient temperature. Theuse of a cooling model will increase the certainty of reaching the desired level of compaction by providing an estimate of time to reach a temperature below which compaction cannot be achieved. Utilizing the concept presented in this study, a highway agency could develop a cooling model that fits into local practices.

In developing the cooling model, the thermal properties of HMA are considered as homogeneous without spatial variation, and the heat transfer is assumed to be one dimensional along the vertical direction. The cooling time is also assumed to be unaffected by changing moisture conditions. In general, the use of thermal properties that are not corrected for changing moisture conditions is considered to furnish good prediction results provided the moisture changes are small.

To verify the applicability of the cooling model, the times available for compaction obtained from the laboratory and the field site are compared with those calculated from the proposed model. The predicted values based on Eq. 3 match well the measured values, as shown in Figure 4.15. Close agreement between the measured and predicted cooling times indicates that the assumptions made during the model development are sufficiently correct.

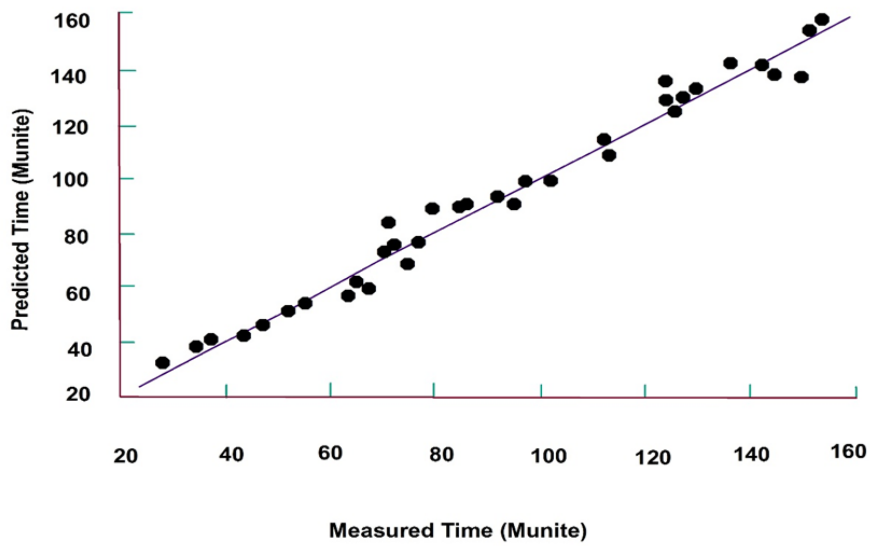

Figure: 4.15. Comparisons of measured and predicted times available for specimen to cool to 80°

Average differences between the measured cooling times and predicted cooling times are approximately 3 min. The fact that laboratory conditions are often in a controlled process implies that the proposed model cannot completely reflect the complex variables that occur in the field. Nevertheless, air temperature, void, and lift thickness can be used to develop a simple model that could consider local situations and reasonably estimate the time available for asphalt pavement compaction.

The cooling model for predicting available compaction time can be developed by considering only the significant factors affecting asphalt pavement construction. Available compaction times predicted by the cooling model are comparable with those obtained from the laboratory and the field. Input data including air temperature, lift thickness, and air voids can be easily measured and estimated in the field. This model can also be extended to estimate the time required to cool from the laydown temperature to the open-traffic temperature of 50° C. Note that the statistical parameters of the model are applicable only to the environmental conditions and materials used in this study. The accuracy of the cooling time predicted by use of the proposed model depends mainly on the quality of the input data and not the statistical parameters of solution.

CHAPTER FIVE

CONCLUSION AND RECOMMENDATION

5.1 Conclusions

Based on the results and analyses, the following conclusionscould be drawn:

1. The cooling rate between dense-graded and porous asphalt mixes is significantly different. The cooling time to reach 80° C is reduced by 50% with an increase in air voids from 10 to 20%. The high air void content in a porous asphalt mixture leads a rapid cooling rate when compared to that in a dense-graded one;

2. The cooling rates at depths of 0, 2.5, and 5 cm are different initially. The rapid cooling rate on the pavement surface is affected by heat transfer that is primarily related to the temperature difference between hot-mix asphalt mixture and air.

The cooling rate at 5 cm depth is relatively low because heat transferred is controlled predominantly by conduction. As the cooling process continues, the cooling rate becomes stable and reaches thermal equilibrium;

3. Increasing the lift thickness of the layer increases the time available for compaction. The increase in time to cool to 80° C appears to be limited when the lift thickness increases from 10 to 15 cm;

4. The time required to cool to 80° C cessation temperature and 50° C open to traffic temperature is determined for variouspavement thicknesses at different depths. The cooling rate measured in the field is comparable with that measured in thelaboratory; and

5. A cooling model is developed to predict the time available for compaction on the basis of lift thickness and ambient temperature. This model gives reasonable estimates of available compaction time during the low temperature time paving operation.

This method provides a quick and efficient way to determine the compaction time with sufficient accuracy. The implication of this field-usable method is that a cooling model can be developed to take into account local conditions by collecting easily acquired data.

5.2 Recommendation

Based on the findings from both literature review and experimental work presented in this study, the following recommendations are made

1. The regression model that is used for predicting the time required to cool to minimum temperature allowed for compaction should bevalidatedusing other modelling tools.

2. The model developed in this study is only applicable to Kaduna State and its environs, therefore there is need to carry out similar studies in other regions of Nigeria in order to come up with models for the whole country.

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