An-Najah National University
Faculty of Graduate Studies

Application of Superpave System for Binder Selection Based on Local Conditions

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DEDICATION

To the owner of the glowing hearts and burning vigor …………………….

To all martyrs of Palestine………………………………………………….

To all who loved Palestine as a home land and Islam as faith a way of life...

To those who provide me with their support to achieve this thesis successfully……………………………………………………………………

To the spirit of my father ………………………………………..............

To my mother, my brother, my sisters, and all my friends………………….

To my teachers who did all their best in helping me to finish this thesis…..

To all of them,

I dedicate this work

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Thanks to God for the blessing granted to us………………………………

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إقرار

أنا الموقع أدناه مقدم الرسالة التي تحمل العنوان:

Application of Superpave System for Binder Selection Based on Local Conditions

تطبيق نظام سوبرباف لاختيار لاصق الإسفلت المناسب بالاعتماد على الظروف المحلية

أقر بأن ما اشتملت عليه هذه الرسالة إنما هي نتاج جهدي الخاص، باستثناء مـــن تمــت الإشارة إليه حيثما ورد، وأن هذه الرسالة ككل، أو أي جزء منها لم يقدم من قبل لنيل أية درجة أو لقب علمي أو بحثي لدى أية مؤسسة تعليمية أو بحثية أخرى.

Declaration

The work provided in this thesis, unless otherwise referenced, is the researcher's own work, and has not been submitted elsewhere for any other degree or qualification.

Student's Name: .... :اسم الطالب

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Date: .... :التاريخ

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Application of Superpave System for Binder Selection Based on Local Conditions

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Abstract

This thesis generally, aims to apply the Superpave system in West Bank regions. This system was developed in the U.S.A during period 1987-1993. The temperature data for West Bank regions was obtained from Directorate of Meteorology at the Ministry of Transportation. The latitude for each city was obtained from geographical maps.

The analysis of data showed that most of Palestinian areas require one Type of binder which is PG 64-10 excluding Jericho, which requires PG 70-10.

According to a study conducted in Jordan about the properties of local binders, which is the same type of binder used in the West Bank, it appeared that it has the same properties of PG 64-16. Therefore, the local binder can be used in all West Bank regions excluding Jericho.

In this thesis several special cases were studied as such as slow traffic, standing traffic, and heavy traffic volume. The optimum binders for these cases were determined accordingly.

Finally, it is recommended to use Superpave system in the West Bank because it has better performance over the Marshall mix design, as found by several studies.

CHAPTER ONE

INTRODUCTION

CHAPTER ONE

INTRODUCTION

1.1 Background

In the last period, the pavement problem appeared clearly in the West Bank cities as the pavement layers could not serve for the entire design period.

This problem is apparent in the projects, which were constructed recently. The deformation and cracks appear in these roads few years after construction. These problems occurred because of several reasons such as defects in the materials, lack of budget for implementing standard project for long design periods, the construction the roads without adhering to the specification and limited professional supervision.

These problems result in adverse effects on the environmental and economical aspects.

In terms of the economical sides, the short period of life of these roads need additional budgets for rehabilitation. On other hand, the bad road conditions decrease its level of service. This causes extra fuel consumption due to decrease in speed and the additional travel time.

From the environmental side, the bad level of service for road due to deformation and cracks cause the vehicles to reduce their speed, so the emission of toxic gases increases.

The bad asphalt performance in the West Bank is also due to using traditional methods for job mixing and binder selecting. Also in many cases there isn't enough monitoring for these traditional methods.

In the U.S.A, research was developed to create a system for comprehensive design of asphalt pavement. Developing of the Superpave method was the major result of research conducted under the Strategic Highway Research Program (SHRP) during 1987-1993.

The research leading to the development of this new system was initiated because previous system tests had difficulty to relate the result obtained from the laboratory to the performance of pavement without field experience as the test performed at standard test temperature. However the Superpave system, which is a shortened from superior performing asphalt pavements, includes method for specifying asphalt binders, mineral aggregate, asphalt mixing design, and a procedure for analyzing and predicting pavement performing.

1.2 Problem Statement

Since there is shortcoming in the performance of the traditional binder and Marshall mix design method, there is a need to adopt new system to suite the local environmental conditions. So, the problems which occur because of traditional method will be treated. For example, there is no test method for asphalt binder stiffness at low temperatures to control thermal cracking. Furthermore, the Marshall and Superpave aggregate gradation requirements for asphalt mixes are different; therefore, yielding different properties. As such, a new system that covers the entire range of pavement temperatures experienced at a particular site is needed.

1.3 Objectives of the Study

This research aims to assist in putting principles of new methodology for the assessing properties of asphalt that should be used in the West Bank based on the Superpave method. Therefore, this thesis works towards selecting the appropriate binder based on local conditions using the Superpave method so as to be applied in the West Bank.

This involves the ability to determine the optimum binder for every region depending on temperature and geographical site, also the job mix design, should be applied according to Superpave requirements.

1.4 Thesis Outline

This thesis contains five chapters, which are summarized as follows. Chapter One presents the introduction, background, problem statement, and objectives.

Detailed review of different studies and researches about developing local criteria for applying Superpave system in many countries is presented in Chapter Two. This chapter includes background of asphalts and its properties and a comparison between traditional method and Superpave system, which was presented in details in this chapter.

The methodology of developing local criteria to select optimum binder for different regions by using specification and requirements of Superpave system is presented in Chapter Three.

Chapter Four presents the results and analysis. The optimum binder for each region in the West Bank is determined using the Superpave method in this chapter. Modifications related to applying these binders under specific conditions are presented; also the effect of using various reliability factors was evaluated. Furthermore, comparison between local gradation and Superpave gradation specifications are presented.

Chapter Five provides conclusions and recommendations of this study.

CHAPTER TWO

LITERATURE REVIEW

CHAPTER TWO

LITERATURE REVIEW

2.1 Introduction

Superpave method was one of the principal results of Strategic Highway Research program (SHRP), which was undertaken in the USA to improve material selection and mixture design by developing a new mix design method that accounts for traffic loading and environmental conditions. Also, new methods of mixture analysis and asphalt binder evaluations were developed.

The literature search showed limited research that covers the selection of binder based on the Superpave method. Below is a summary of search results related to this subject.

General information about asphalt mixes and Superpave method is presented in Appendix A

2.2 Asphalt Binder Evaluation

Asphalt selection for the Superpave mix design is performance based and dependent on climatic and traffic conditions. The high and low temperature requirement of the binder differentiates among the various grades of binders, for example, an asphalt binder grade PG58-28 means that the asphalt must meet high temperature requirements of 58 °C and law temperature requirement of -28C^{o}.

Once a designer selects grade based on temperature the grade may be adjusted for different loading conditions (Ksaibati and Stephen, 1998).

2.2.1 Design Pavement Temperature

The Superpave mix design method determines both high and low design pavement temperature. These temperatures are determined as follows:

- High pavement temperature - based on the 7-day average high air temperature of the surrounding area.
- Low pavement temperature- based on the 1-day low air temperature of the surrounding area.

Using these temperatures as a starting point, Superpave then applies a reliability concept to determine the appropriate PG asphalt binder. PG asphalt binders are specified in 6°C increments.

Table (2.1): Prediction of PG Grades for Different Crude Oil Blends

Low Temperature, °C	High Temperature, °C 52	58	64	70	76
-16	52-16	58-16	64-16	70-16	76-16
-22	52-22	58-22	64-22	70-22	76-22
-28	52-28	58-28	64-28	70-28	76-28
-34	52-34	58-34	64-34	70-34	76-34
-40	52-40	58-40	64-40	70-40	76-40

= Crude Oil

= High Quality Crude Oil

= Modifier Required

*(Source: Washington State DOT, 2007)

2.2.2 Design Pavement Temperature Adjustments

Design pavement temperature calculations are based on HMA pavements subjected to fast moving traffic. Specifically, the Dynamic Shear Rheometer (DSR) test is conducted at a rate of 10 radians per second, which corresponds to a traffic speed of about 90 km/hr (55 mph) (Washington State DOT, 2007)[(1)].

Pavements subject to significantly slower (or stopped) traffic such as intersections, toll booth lines and bus stops should contain a stiffer asphalt binder than that which would be used for fast-moving traffic. Superpave allows the high temperature grade to be increased by one grade for slow transient loads and by two grades for stationary loads. Additionally, the high temperature grade should be increased by one grade for anticipated 20-year loading in excess of 30 million ESALs.

For pavements with multiple conditions that require grade increases only the largest grade increase should be used. Therefore, for a pavement intended to experience slow loads (a potential one grade increase) and greater than 30 million ESALs (a potential one grade increase) the asphalt binder high temperature grade should be increased by only one grade. Table 2.2 shows two examples of design high temperature adjustments - often called "binder bumping".

[(1)] The literature review in this thesis depended widely on the Washington State DOT (WSDOT) source as specified in several instances in this chapter. However, the (WSDOT) mainly adopts the Asphalt Institute (AI) method, which is the primary source for the Superpave system, and specifies

Table (2.2): Examples of Design Pavement Temperature Adjustments for Slow and Stationary Loads

Original Grade	Grade for Slow Transient Loads (increase 1 grade)	Grade for Stationary Loads (increase 2 grades)	20-yr ESALs > 30 million (increase 1 grade)
PG 58-22	PG 64-22	PG 70-22	PG 64-22
PG 70-22*	PG 76-22	PG 82-22	PG 76-22

Source: Washington State DOT 2007

The highest possible pavement temperature in North America is about 70°C but two more high temperature grades were necessary to accommodate transient and stationary loads.

WSDOT Washington's State Department of Transportation (WSDOT) uses the following guidance shown in the Table 2.3 when considering adjustments to the design high temperature of a PG asphalt binder (sometimes referred to as "binder bumping").

2.3 Comparison between PG system and AC and AR system

Table 2.4 shows how the Superpave PG system addresses specific penetration, AC and AR grading system general limitation.

Table (2.3): Design Pavement Temperature Adjustments ("Binder Bumping")

Situation	Adjustment to High Temperature Grade
15-year design ESALs of 10-30 million	Consider Increasing 1 Grade
15-year design ESALs ≥ 30 million	Increase 1 Grade
Slow Traffic (10 - 45 mph)	Increase 1 Grade
Standing Traffic (0 - 10 mph)	Increase 2 Grades

Source: Washington State DOT, 2007**)**

Additionally, all mountain passes should use a base grade of PG 58-34.

2.4 Studies and Research

A limited number of researchers studied the development of the Superpave criteria over the years. Research conducted in Jordan (Asi, 2005) was directed toward performance evaluation of Superpave and Marshall mix design to suite Jordan climatic and traffic conditions. In this research, a comprehensive evaluation of the locally available aggregate usually used in the asphalt concrete mixtures was carried out to ensure that these materials conform to the new mix design procedure developed by Superpave.

Table (2.4): Prior Limitations vs. Superpave Testing and Specification Features

Limitations of Penetration, AC and AR Grading Systems	**Superpave Binder Testing and Specification Features that Address Prior Limitations**
Penetration and ductility tests are empirical and not directly related to HMA pavement performance.	The physical properties measured are directly related to field performance by engineering principles.
Tests are conducted at one standard temperature without regard to the climate in which the asphalt binder will be used.	Test criteria remain constant, however, the temperature at which the criteria must be met changes in consideration of the binder grade selected for the prevalent climatic conditions.
The range of pavement temperatures at any one site is not adequately covered. For example, there is no test method for asphalt binder stiffness at low temperatures to control thermal cracking.	The entire range of pavement temperatures experienced at a particular site is covered.
Test methods only consider short-term asphalt binder aging (thin film oven test) although long-term aging is a significant factor in fatigue cracking and low temperature cracking.	Three critical binder ages are simulated and tested: 1. Original asphalt binder prior to mixing with aggregate. 2. Aged asphalt binder after HMA production and construction. 3. Long-term aged binder.
Asphalt binders can have significantly different characteristics within the same grading category.	Grading is more precise and there is less overlap between grades.
Modified asphalt binders are not suited for these grading systems.	Tests and specifications are intended for asphalt "binders" to include both modified and unmodified asphalt cements.

(Source: Washington State DOT, 2007)

A performance grading map was generated to the Hashemite Kingdom of Jordan. In this map the country was divided to different zones according to the highest and lowest temperature ranges that the asphalt might be subjected to. Using local (Jordanian) material, loading and environmental conditions, a comprehensive study of the performance of the two mixes designed using the Superpave and Marshall mix design procedure was carried out in this research.

Samples from both mixes were prepared at the design asphalt contents and aggregate gradation and were subjected to a comprehensive mechanical evaluation testing. These tests included Marshall stability, loss of Marshall stability, indirect tensile strength, loss of indirect tensile strength, resilient modulus, fatigue life, rutting, and creep. In all performed tests, Superpave mixes proved their superiority over Marshall mixes.

The conclusions of Asi's research and its experiments were

- In general, the performance grade of locally produced asphalt is PG64-16.
- A temperature zoning map was developed for the Hashemite Kingdom of Jordan; it consisted of three grade zones, PG64-10, PG64-16, and PG70-10.
- Jordanian produced asphalt can be used without modification in all parts of Jordan except Aqaba, Ruwaishied, and Ghorsafi. In these areas, asphalt grade should be modified to shift its grade to PG70-10. This modification might just require air blowing of local asphalt.

- Local aggregates met both Superpave consensus properties and source properties.

- Locally (Jordanian) used aggregate gradation was not suitable according to the Superpave mix design procedure.

- Superpave mix design procedure recommended for the local environmental and loading conditions lower asphalt content than that predicted by Marshall mix design procedure. This might explain the causes behind the bleeding asphalt concrete surfaces and some of the distresses common in the local asphalt structure.

- Superpave showed superior performance over Marshall mix .

A Norwegian research project (PROKAS) was conducted in the period 1998- 2004. One objective was to develop performance-based specification for Norwegian asphalt mixtures. As part of this project, a new binder selection system was sketched (Lerfald, 2004).

This system was based on the result from the Norwegian research project (New Asphalt Technology); a project aiming at adoption of the Superpave binder system to Norwegian conditions. An attempt to verify this binder selection system in the laboratory was made. Deformation properties of asphalt mixtures with different binder stiffness were measured (Lerfald, 2004). The same aggregate and material grading was used in all samples. The new asphalt binder selection system showed relatively good correlation results from deformation test. However, further testing with other mixtures and higher temperatures were required to fine tune and fully verify the system. The main conclusions from the investigations were:

- The indentation repeated axial test (INDENT) on the Nottingham Asphalt Tester (NAT) seems to rank asphalt mixes with the same aggregate and varying binder stiffness in reliable way.

- Result from deformation measurements of asphalt of asphalt mixes with different binder grades seemed to correlate relatively well with the new proposed system for binder selection base on local climate and traffic loading.

CHAPTER THREE

METHODOLOGY

CHAPTER THREE

METHODOLOGY

3.1 Introduction

This thesis aims at developing local criteria for selecting optimum binder for paving mixture in the Palestinian regions according to the Superpave system as there is a great difference in temperature between Palestinian regions.

It should be noted that the laboratory equipment needed for the various Superpave tests are not available in the West Bank. As such, various types of analyses and laboratory testing and verification were not possible. Therefore, the methodology presented below was designed taking into consideration the unavailability of lab equipments to conduct various Superpave tests.

3.2 Work Procedure

The following represents the methodology by which the above aim can be achieved in order to study the application of Superpave system in Palestine.

3.2.1 Collecting Data

Weather data (temperatures) was obtained from Department of Meteorology. The Department has eight weather stations that are distributed across the West Bank.

On principle, the collected data should cover a minimum of 10 years of continuous temperature recording. The data are then analyzed to obtain the yearly average hottest seven days, minimum recorded air temperature, in addition to the standard deviation at all stations.

The location (latitude in degrees) of all stations was obtained from the geographical maps.

3.2.2 Determining Maximum and Minimum Pavement Temperature

The Superpave system considers that the pavement temperature, not the air temperature, should be used as the design temperature. Therefore, equations developed by the Superpave system were used to convert maximum air temperature to maximum design pavement temperature.

The low pavement design temperature can be used as either the low air temperature, which is rather conservative or it can be determined from the low air temperature using equation developed for the Superpave system.

3.2.3 Selecting Asphalt Binder

After calculating high and low pavement design temperatures, asphalting binder grades were selected based on 98% reliability from tables developed by Superpave system for this purpose. This gives a listing of more commonly used asphalt binder grades with their associated physical properties.

3.2.4 Testing Cases of Various Reliability Factors

The effect of using various reliability factors on the selection of binder type was tested by changing the reliability factor (Z) in the low and high design pavement design temperature equation.

3.2.5 Comparing Gradation Systems

As the Marshall and Superpave systems adopt different gradation requirements, a comparison of specifications between local gradation, which is based on Marshall, and Superpave gradation was presented.

3.2.6 Adjusting Binder Grade for Traffic Speed and Loading

The selected binder was adjusted for other traffic conditions different from that assumed in the previous procedure.

3.2.7 Constructing Maps

West Bank was divided into several temperature zones. The asphalt grade for each zone was displayed on the West Bank map.

CHAPTER FOUR

RESULTS AND ANALYSIS

CHAPTER FOUR

RESULTS AND ANALYSIS

4.1 Introduction

This chapter aims at analyzing the data which, was obtained from the General Directorate of Meteorology at the Ministry of Transport, to divide the West Bank into different temperature zones so the optimum binder grade for each zone could be determined.

4.2 Derivation of Temperatures Limits

The high and low design pavement temperatures were determined from equations developed by the Superpave system depending on maximum and minimum air temperatures and latitude of the intended region. In addition, equation (4.1) was used to convert maximum air temperature to maximum design pavement temperature.

$$\mathbf{T_{20mm} = [(T_{air} - .00618Lat^2 + 0.2289Lat + 42.2) * (0.9545)] - 17.78} \ldots\ldots \text{(Eqn. 4.1)}$$

(Source: Asphalt Institute, 2001)

where:

T_{20mm} : high pavement design temperature at depth of 20mm.

T_{air} : seven day average high temperature.

Lat : geographical latitude of the project location in degrees.

The low pavement design temperature can be used as either the low air temperature, which is rather conservative or it can be determined from the low air temperature using equation 4.2.

$$\mathbf{T_{pav} = 1.56 + 0.72T_{air} - 0.004Lat^2 + 6.26Log_{10}(H+25) \_ Z(4.4+0.5\sigma_{air}{}^2)^{0.5}} \quad \text{…(Eqn 4.2)}$$

(Source: Garber and Hoel, 2002)

where:

T_{pav}: low AC pavement temperature below surface C^o.

T_{air}: low air temperature C^o.

Lat: latitude of the project location in degrees.

σ_{air}: standard deviation of the mean low temperature.

Z: from standard normal distribution table, Z= 2.055 from 98% Reliability.

The standard deviations are determined to use the reliability concept in selecting the design pavement temperatures. For example, consider mean seven–day maximum air temperature is $35C^o$ with standard deviation $1C^o$. The probability that during a year the seven day maximum temperature will exceed $35C^o$ is 50 percent, but only 2 percent chance for it to exceed $37C^o$.

So selecting maximum air temperature of 37 C^o will achieve reliability of 98% that the maximum air temperature will not be exceeded.

98% reliability = mean ± (2* standard deviation).

Table 4.1 was developed through Superpave research to select optimum binder according minimum and maximum pavement temperature determined by the previous equations. For example, consider the maximum design pavement temperature was $62C^o$ and the minimum design pavement temperature was -12. As the high temperature is more than $58C^o$ and less than 64C^{o} so the high temperature grade will be 64 in the other hand the minimum design pavement temperature is between -10 and -16 so the low design grade will be -16 as a result the optimum binder will be PG 64-16, also the intended table listing associated physical properties for the commonly used asphalt binder grades.

Table (4.1): Performance Graded Asphalt Binder Specifications

Performance Grade	PG 46			PG 52							PG 58					PG 64					
	34	40	46	10	16	22	28	34	40	46	16	22	28	34	40	10	16	22	28	34	40
Average 7-day Maximum Pavement Design Temperature, °C[a]	< 46			< 52							< 58					< 64					
Minimum Pavement Design Temperature, °C[a]	-34	-40	-46	-10	-16	-22	-28	-34	-40	-46	-16	-22	-28	-34	-40	-10	-16	-22	-28	-34	-40
ORIGINAL BINDER																					
Flash Point Temp, T 48, Minimum (°C)	230																				
Viscosity, ASTM D 4402:[b] Maximum, 3 Pa*s, Test Temp, °C	135																				
Dynamic Shear, TP 5:[c] G*/sinδ[f], Minimum, 1.00 kPa Test Temp @ 10 rad/s, °C	46			52							58					64					
ROLLING THIN FILM OVEN RESIDUE (T 240)																					
Mass Loss, Maximum, percent	1.00																				
Dynamic Shear, TP 5: G*/sinδ[f], Minimum, 2.20 kPa Test Temp @ 10 rad/s, °C	46			52							58					64					
PRESSURE AGING VESSEL RESIDUE (PP 1)																					
PAV Aging Temperature, °C[d]	90			90							100					100					
Dynamic Shear, TP 5: G*/sinδ[f], Maximum, 5000 kPa Test Temp @ 10 rad/s, °C	10	7	4	25	22	19	16	13	10	7	25	22	19	16	13	31	28	25	22	19	16
Physical Hardening[e]	Report																				
Creep Stiffness, TP 1 Determine the critical cracking temperature as described in PP 42	-24	-30	-36	0	-6	-12	-18	-24	-30	-36	-6	-12	-18	-24	-30	0	-6	-12	-18	-24	-30
Direct Tension, TP 3 Determine the critical cracking temperature as described in PP 42	-24	-30	-36	0	-6	-12	-18	-24	-30	-36	-6	-12	-18	-24	-30	0	-6	-12	-18	-24	-30

Table 4.1, Continued

Performance Grade	PG 70						PG 76					PG 82				
	10	16	22	28	34	40	10	16	22	28	34	10	16	22	28	34
Average 7-day Maximum Pavement Design Temperature, °C[a]	< 70						< 76					< 82				
Minimum Pavement Design Temperature, °C[a]	-10	-16	-22	-28	-34	-40	-10	-16	-22	-28	-34	-10	-16	-22	-28	-34
ORIGINAL BINDER																
Flash Point Temp, T 48, Minimum (°C)	230															
Viscosity, ASTM D 4402:[b] Maximum, 3 Pa*s, Test Temp, °C	135															
Dynamic Shear, TP 5:[c] $G^*/\sin\delta$[f], Minimum, 1.00 kPa Test Temp @ 10 rad/s, °C	70						76					82				
ROLLING THIN FILM OVEN RESIDUE (T 240)																
Mass Loss, Maximum, percent	1.00															
Dynamic Shear, TP 5: $G^*/\sin\delta$[f], Minimum, 2.20 kPa Test Temp @ 10 rad/s, °C	70						76					82				
PRESSURE AGING VESSEL RESIDUE (PP 1)																
PAV Aging Temperature, °C[d]	100 (110)						100 (110)					100 (110)				
Dynamic Shear, TP 5: $G^*\sin\delta$[f], Maximum, 5000 kPa Test Temp @ 10 rad/s, °C	34	31	28	25	22	19	37	34	31	28	25	40	37	34	31	28
Physical Hardening[e]	Report															
Creep Stiffness, TP 1 Determine the critical cracking temperature as described in PP 42	0	-6	-12	-18	-24	-30	0	-6	-12	-18	-24	0	-6	-12	-18	-24
Direct Tension, TP 3 Determine the critical cracking temperature as described in PP 42	0	-6	-12	-18	-24	-30	0	-6	-12	-18	-24	0	-6	-12	-18	-24

Source: Washington DOT, 2007.

a Pavement temperatures are estimated from air temperatures using an algorithm contained in the LTPP Bind program, may be provided by the specifying agency, or by following the procedures as outlined in MP 2 and PP 28.

b This requirement may be waived at the discretion of the specifying agency if the supplier warrants that the asphalt binder can be adequately pumped and mixed at temperatures that meet all applicable safety standards.

c For quality control of unmodified asphalt binder production, measurement of the viscosity of the original asphalt binder may be used to supplement dynamic shear measuremments of $G^*/\sin\delta$ at test temperatures where the asphalt is a Newtonian fluid.

d The PAV aging temperature is based on simulated climatic conditions and is one of three temperatures 90°C, 90°C or 110°C. The PAV aging temperature is 100°C for PG 58- and above, except in desert climates, where it is 110°C.

e Physical hardening -- TP 1 is performed on a set of asphalt beams according to Section 12, except the conditioning time is extended to 24 hours ± 10 minutes at 10°C above the minimum performance temperature. The 24-hour stiffness and *m*-value are reported for information purposes only.

f $G^*/\sin\delta$ = high temperature stiffness and $G^*/\sin\delta$ = intermediate temperature stiffness

4.2 Analysis of Data for Each Region

The data for seven regions in the West Bank was obtained from Department of Meteorology at the Ministry of Transport. As indicated earlier, the Superpave system requires collecting temperature data for 10 years. Such a database was not available for West Bank regions due to various reasons; one of which is the on-going political conditions that prohibited the continuation of various public services, such as temperature data collection.

Obtaining data from other sources was difficult since the daily temperatures for each year were needed to determine the hottest seven day temperatures, which may occur at one month. However, nearly all sources of data just provide monthly or yearly data. Therefore, such data could not be used for the purpose of this study.

Although there was a gap in the data for some study regions because of the political situations during that period, which restricted collecting comprehensive data over ten years; however, the scattering of available data was small. Therefore, it was concluded that the available data can be used with reasonable accuracy. Collected temperature data for all study regions are presented in Appendix B.

4.2.1 Ramallah City

The data, which was obtained from the General Directorate of Meteorology for Ramallah City represents the hottest seven-day period for each year and the minimum temperatures for the same periods. Tables 4.2 and 4.3 show temperatures data covered the period from 2000 to 2006, excluding the year 2005.

Table (4.2): Hottest Seven Days in Ramallah Each Year C^o

2000	2001	2002	2003	2004	2005	2006
37.5	35.4	39.4	33.6	35.6	NA	32.2
37.0	34.5	36.8	33.6	35.0	NA	30.1
34.2	34.4	36.5	33.5	33.2	NA	29.6
34.0	33.5	36.0	33.2	32.8	NA	29.5
34.0	32.2	35.6	33.0	32.8	NA	29.4
34.0	32.0	35.5	32.9	31.0	NA	29.4
33.6	31.8	35.0	32.8	31.0	NA	28.5

Table (4.3): Minimum Temperatures in Ramallah C^o

2000	2001	2002	2003	2004	2005	2006
Min	**Min**	**Min**	**Min**	**Min**	**Min**	**Min**
-1.0	2.8	NA	0.0	0.0	NA	1.4

The average of maximum temperatures $T_{(average\ max)} = (\Sigma Ti)/N$

where:

T : hottest seven-day temperature

N: the number of observed value

So T(average) = 1405.6/42= 33.5C^o.

Standard deviations= $\sqrt{(\Sigma(Ti-T_{(average)})^2/(N-1))}$.

S=$\sqrt{(237.88/41)}$ = 2.41C^o

The same procedure will be applied to the minimum temperatures data to determine average and standard deviation for that data.

$T_{(average\ min)}$= 3.2/5= 0.64C^o

S_{min}= $\sqrt{(8.752/4)}$= 1.5C^o

The analysis of the data showed that it was scattered as the standard deviation of the maximum temperatures was 2.41, but the standard deviation for the minimum temperatures data was 1.5.

From these results, high design pavement temperature was determined using equation 4.1 with a latitude value 31.9^{o}N. The value of the determined temperature was $60C^{o}$ (using 98% reliability).

The lowest temperature over the study period (10 years) did not go below $-4C^{o}$. Furthermore, the low design pavement temperature equation (eqn. 4.2) always gives values higher than the lowest air temperature. However, the performance grade selection table (Table 4.1) specifies the highest value of lowest temperatures for the binder as $-10C^{o}$. Therefore, the atmospheric temperature could be used as low design pavement temperature, which gives conservative result; this will be $-2.3C^{o}$ (using 98% reliability).

From the minimum and high design pavement temperatures, using Table 4.1, the optimum binder for Ramallah City will be PG 64-10.

4.2.2 Nablus City

Nablus stands in the middle of the West Bank on latitude $32.23N^{o}$. The available data in the General Directorate of Meteorology covers the period between 1998 and 2006 excluding 2005 for the maximum air temperatures. However, the minimum air temperatures for 2002, 2003, and 2005 were not available because of the political conditions during that period.

It was noted that the average for the yearly hottest seven days in that period was 35.3, and the scattering of data of was less than the data of Ramallah City. This can be known from standard deviation value, which was 1.6 for the maximum temperatures.

On the other hand, the yearly low temperatures for the same period was 1.7 C^o, the standard deviation for these tempretures was 1.71, which indicates that the low temperature data was scattered more than those for Ramallah City.

Using equations 4.1 and 4.2, the high design pavement temperature was 60.3C^o, and the low one was -1.7C^o (using 98% reliability).

As it was mentioned before, the low air temperature could be conservative in determine the optimum binder, instead of using low design pavement temperature equation.

Table 4.1 shows that the suitable binder is PG 64-10 for Nablus City depending on the low and high pavement temperatures.

4.2.3 Hebron City

The registered data for Hebron City were for nine years between 1998-2006 excluding year 2004.

The standard deviation value was 1.71. Therefore, the scattering of Hebron data for the maximum temperature was more than Nablus City and less than Ramallah City. Also scattering of low temperatures data for Hebron City was less than previous two cities. This can be noted from the value of the standard deviation of 1.39.

The average of the yearly hottest seven days temperatures for that period is 33.2, also the minimum temperature for the same period is -0.733 C^o.

The high design pavement temperature was found to be 58.6C^o using equation 4.1 and 98% reliability.

Because the low air temperature is not less than -10, so it could be considered as low design pavement temperature, based on 98% reliability and a low temperature of -3.5C^o. Referring to Table 4.1, the optimum binder for Hebron City was found to be PG 64-10.

4.2.4 Tulkarm City

Tulkarm lies in the north of West Bank at latitude 32.32 N^o. The data which has been received from the General Directorate of Meteorology was taken for ten years from 1998-2007, but data for of the period of 2002-2003 was not available.

It is noticeable that the data was scattered as the standard deviation of the maximum temperatures was 2.0, and for minimum temperatures it was 2.14.

The average yearly hottest seven-day temperature for that period was 37.1C^o and the lowest temperature for the same period was 4.5.

According to the previous information, the high and low design pavement temperatures could be found by using equations 4.1 and 4.2. As the lowest air temperature is more than -10, so according to Table 4.1, the air temperature could be considered as low design pavement temperature.

Using 98% reliability, the high and low design pavement temperatures will be 62.8C^{o} and 0.2C^{o}, respectively.

Referring to Table 4.1 and by using the high and low design pavement temperatures, the optimum binder of Tulkarm City is PG 64-10.

4.2.5 Jenin City

Jenin City lies on latitude 32.47N^{o} in the north of the West Bank. The data which is taken from the General Directorate of Meteorology was for five years only because of the Israeli invasions of the West Bank in general and Jenin City in particular; the data for the periods of 2002-2003 and 2005-2006 is not available.

Although the number of years is not sufficient, the scattering of the data is less than the previous cities for maximum temperatures, as the standard deviation of the maximum temperatures was 1.34. But it was noticeable that the low air temperature data was more scattered as the standard deviation for the low temperatures was 2.39.

In order to find out the optimum binder for Jenin City, the average of the yearly hottest seven days was determined for that period with the result 38.6C^{o}; the lowest temperatures for the same period was 2.76.

Using equations 4.1 and 4.2, high and low design pavement temperatures can be obtained. The high design pavement temperature was 62.9C^{o} while the low design pavement temperature can be considered as the lowest air temperature, which is -2 because it was more than -10C^{0}. These values were obtained using 98% reliability for temperatures.

According to Table 4.1, the optimum binder for this city is PG 64-10.

4.2.6 Maythalon Region

Maythalon lies on latitude 32.35N^o in the north of West Bank.

The recorded data for Maythalon was taken for seven years from 1998 to 2007 excluding the data of 2002, 2003, and 2006, which was not recorded for the same political situations in the West Bank.

Standard deviation for maximum temperatures was 1.37 and it is close to the standard deviation of Jenin. The scattering of the data for the maximum temperatures for Maythalon was less than the other Palestinian cities.

The scattering of data for the low temperatures was less than the previous cities except for Hebron City where the standard deviation for these values was 1.67.

In order to find out the optimum binder, the average of the yearly hottest seven days for that period should be determined, and the result was 37.8C^o. The minimum of air temperatures for the same period was -0.6C^o.

Based on the previous analysis, the high and low design pavement temperatures could be obtained using equations 4.1 and 4.2. It was clear that the high design pavement temperature was 62.2C^o, but the low design pavement temperature was -3.9C^o, which represent low air temperature without using equation 4.2 because the low air temperature was more than -10. These values for high and low design pavement temperatures were achieved based on 98% reliability.

Based on this, the optimum binder for Maythalon City is PG 64-10.

It is noted that the optimum binder for the previous cities is the same because of the similar weather and geographical conditions.

4.2.7 Jericho City

Jericho City is considered a different case relative to the other Palestinian cities based on its climate and geographical conditions.

Jericho has the highest recorded temperatures in the West Bank, and it lies below sea level on latitude 31.85N^o in the middle of the West Bank.

The data was taken through eight years from 1998-2006 excluding the year 2005.

After analyzing the received data, it was realized that the average of the yearly hottest seven days was 43.8, and this is the highest one in the West Bank.

The scattering of the data was acceptable according to the other cities as the standard deviation was 1.45 for the maximum temperature; however, the lowest temperatures were 3C^o.

The scattering of the data for the low air temperatures was the least with respect to the previous cities as the value was 1.18.

The high and low design pavement temperatures were determined using the previous method, and they were 68.2 and 1.9 C^o. These values were based on 98% reliability.

The value of the low design pavement temperatures, which was -3.9 is the value of low air temperature based on 98% reliability without using equation 4.2 for the same previous reason stated for the other cities.

The optimum binder for Jericho City is PG 70-10.

4.3 Summary of Results

Table 4.4 shows a summary for the results of the analyzed data for each city in the West Bank.

Table (4.4): Summary of Results for the Analyzed Data

City	Latitude in Degrees	Max. Air Temp.	Max. Stand. Dev.*	Min. Air Temp.	Min. Stand. Dev. *	High Des. Pav. Temp. (C^o)	High. Des. Tem. C^o (98% Reliability)	Low Des. Pav. Temp. (C^o)	Low Des. Temp. C^o (98% Reliability)	Binder Selection
Ramallah	31.9	33.5	2.408	0.64	1.48	55.44	60.26	0.64	-2.32	PG 64-10
Nablus	32.23	35.3	1.597	1.77	1.71	57.11	60.30	1.77	-1.66	PG 64-10
Hebron	31.54	33.2	1.713	-0.733	1.39	55.21	58.64	-0.73	-3.52	PG 64-10
Tulkarm	32.32	37.1	2.007	4.5	2.14	58.81	62.83	4.50	0.22	PG 64-10
Jenin	32.47	38.6	1.342	2.77	2.39	60.22	62.90	2.77	-2.02	PG 64-10
Maythalon	32.35	37.8	1.373	-0.6	1.67	59.47	62.22	-0.60	-3.94	PG 64-10
Jericho	31.85	43.8	1.448	4.3	1.18	65.28	68.18	4.30	1.93	PG 70-10

* Max. Stand. Dev. and Min. Stand. Dev.: maximum and minimum standard deviation, respectively.

Figure 4.1 shows a map that was drawn to divide the West Bank into different temperature zones. It was found that two asphalt grades are required for West Bank regions; PG 64-10 is suitable for most areas in the West Bank except Jericho region, which requires PG 70-10.

4.4 Cases of Various Reliability Factors

The previous analysis was performed for 98% reliability factor, which is the most used reliability in the literature. This section evaluates the effect of using different reliability factors on the selection of binder type.

The following equations were used to determine high and low design pavement temperature for different reliability factors.

$HDPT = DPT + Z\sigma$ ………………………………………………Eqn 4.3

$LDPT = DPT – Z\sigma$ ………………………………………………Eqn. 4.4

Where:

HDPT : high design pavement temperature with specific reliability factor.

DPT : design pavement temperatures determined by using Eqn. 4.1, 4.2.

Z : from standard normal distribution table (selected values are shown in Table 4.5).

σ : standard deviation of the mean of max and low temperatures.

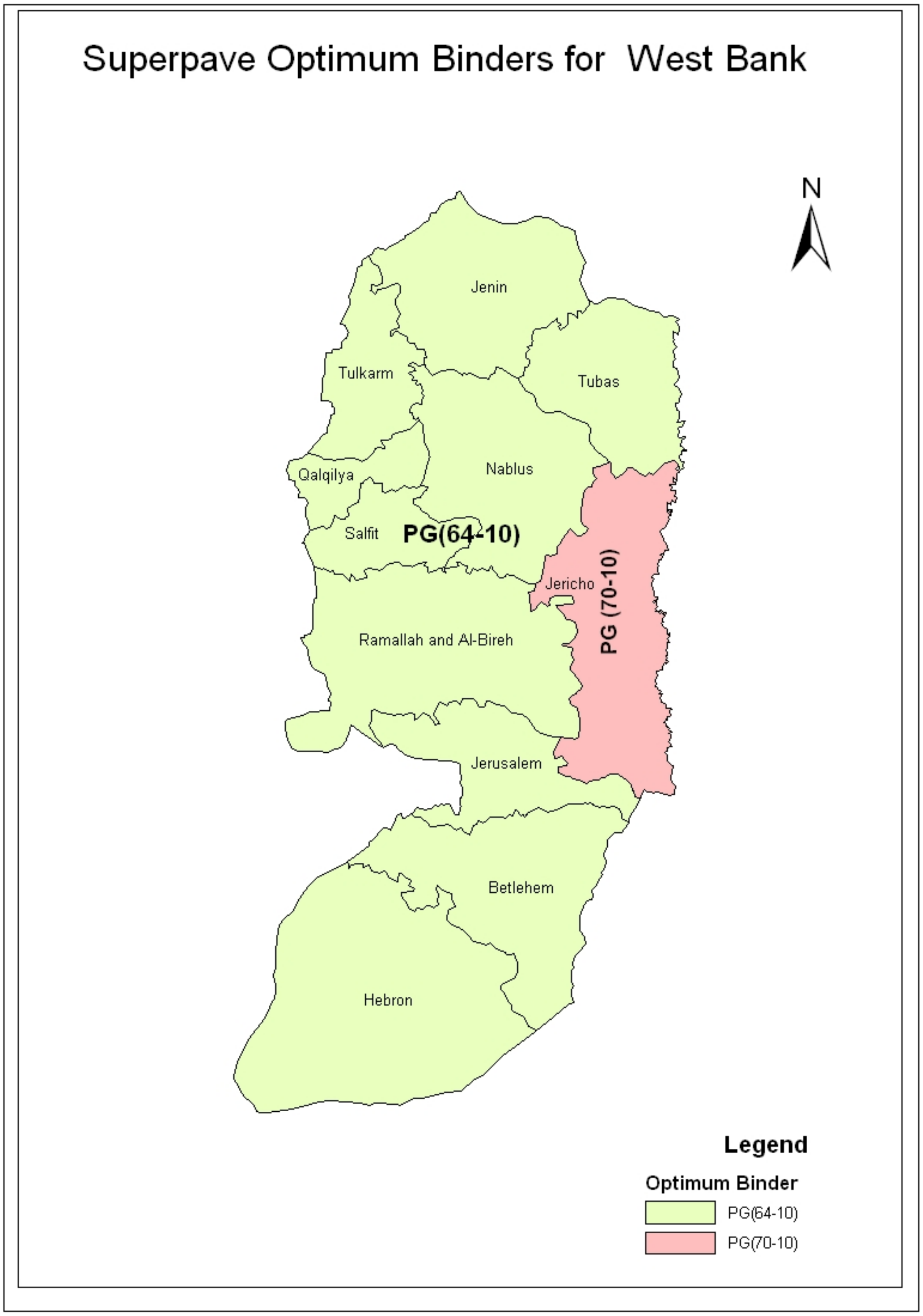

Figure (4.1): Optimum Binders in the West Bank

Table (4.5): Z-Values for Various Reliability Factors

Reliability	Z
90%	1.282
95%	1.655
98%	2.055
99%	2.327

So, the summary of binder selection results for Palestinian regions is shown in the Table 4.6.

It is shown in Table 4.6 that the binder type remained the same for all regions regardless of the reliability factor used, except for Hebron region where binder grade PG(58-10) was appropriate for reliability 90% and 95%, and binder grade PG(64-10) was appropriate for 98% and 99% reliability. One reason for this might be that Hebron has lowest maximum air temperature for all West Bank regions. Since the 98% reliability is the most widely used, and binder grades in West Bank regions did not vary with different reliability factors used (except for Hebron), therefore, the 98% reliability is used in further analysis.

Table 4.6: Summary of Binder Selection for Various Reliability Factors

City	90% Reliability			95% Reliability			98% Reliability			99% Reliability		
	High Des. Tem. C°	**Low Des. Tem. C°**	**Binder Selection**	**High Des. Tem. C°**	**Low Des. Tem. C°**	**Binder Selection**	**High Des. Tem. C°**	**Low Des. Tem. C°**	**Binder Selection**	**High Des. Tem. C°**	**Low Des. Tem. C°**	**Binder Selection**
Ramallah	58.53	-1.25	PG(64-10)	59.43	-1.8	PG(64-10)	60.26	-2.32	PG(64-10)	61.05	-2.80	PG(64-10)
Nablus	59.16	-0.42	PG(64-10)	59.75	-1.07	PG(64-10)	60.30	-1.66	PG(64-10)	60.82	-2.22	PG(64-10)
Hebron	57.41	-2.51	PG(58-10)	58.05	-3.04	PG(58-10)	58.64	-3.52	PG(64-10)	59.20	-3.97	PG(64-10)
Tulkarm	61.38	1.75	PG(64-10)	62.13	0.96	PG(64-10)	62.83	0.22	PG(64-10)	63.48	-0.48	PG(64-10)
Jenin	61.94	-0.30	PG(64-10)	62.44	-1.19	PG(64-10)	62.90	-2.02	PG(64-10)	63.34	-2.80	PG(64-10)
Maythaloon	61.23	-2.74	PG(64-10)	61.75	-3.36	PG(64-10)	62.22	-3.94	PG(64-10)	62.67	-4.48	PG(64-10)
Jericho	67.14	2.78	PG(70-10)	67.68	2.33	PG(70-10)	68.18	1.93	PG(70-10)	68.65	1.54	PG(70-10)

4.5 Adjusting Binder Grade for Traffic Speed and Loading

The previous procedure for selecting asphalt binder is based on an assumed traffic condition consisting of designed number of fast transient load. For other traffic conditions that are different, the speed of loading has additional effects on the ability of the pavement to resist permanent deformation at high temperature conditions.

When there is slow traffic (10-45) mph (15-70) km/hr, the selected asphalt binder based on the procedure described earlier should be shifted one high temperature grade. Also when there is a standing traffic (0-10) mph (0-15) km/hr, the selected binder should be shifted two high temperature grades.

In addition to the shifting, the designer should modify the selected binder for accumulative traffic load. For Equivalent Single Axle Load (ESAL) of 10-30 million, one should consider shifting the selected binder by one high temperature grade. However, for ESAL exceeding 30 million, a shift of one high temperature grade is required.

It should be noted here that the ESAL value of 30 million is much greater than existing ESAL on all West Bank roads.

Table (4.7): Adjustment for Design High Temperature of a PG Asphalt Binder

Situation	Adjustment for High Temperature Grade
15-year design ESALs of 10 - 30 million	Consider Increasing 1 Grade
15-year design ESALs ≥ 30 million	Increase 1 Grade
Slow Traffic (10 - 45 mph) (15-70) km/hr	Increase 1 Grade
Standing Traffic (0 - 10 mph) (0-15) km/hr	Increase 2 Grades

Source: Washington DOT, 2007

Note: all mountain roads should use a base grade of PG 58-34

If these considerations are applied to the Palestinian regions, the following asphalt binders for each case will be obtained (Table 4.8).

Figures 4.2 to 4.5 show maps for West Bank, which were drawn with two temperature zones, and the optimum asphalt binder was selected under the adjustment criteria for speed and loading.

Table (4.8): Adjustment Binder Grade in the West Bank for Traffic Speed and Loading

City	Selected Binder	Shift due slow traffic (10-45) mph	Shift due standing traffic (0-10) mph	Shift due ESAL (10-30) million (considered)	Shift due ESAl>30 million (required)
Ramallah	PG 64-10	PG 70-10	PG 76-10	PG 70-10	PG 70-10
Nablus	PG 64-10	PG 70-10	PG 76-10	PG 70-10	PG 70-10
Hebron	PG 64-10	PG 70-10	PG 76-10	PG 70-10	PG 70-10
Tulkarm	PG 64-10	PG 70-10	PG 76-10	PG 70-10	PG 70-10
Jenin	PG 64-10	PG 70-10	PG 76-10	PG 70-10	PG 70-10
Jericho	PG 70-10	PG 76-10	PG 82-10	PG 76-10	PG 76-10
Maythalon	PG 64-10	PG 70-10	PG 76-10	PG 70-10	PG 70-10

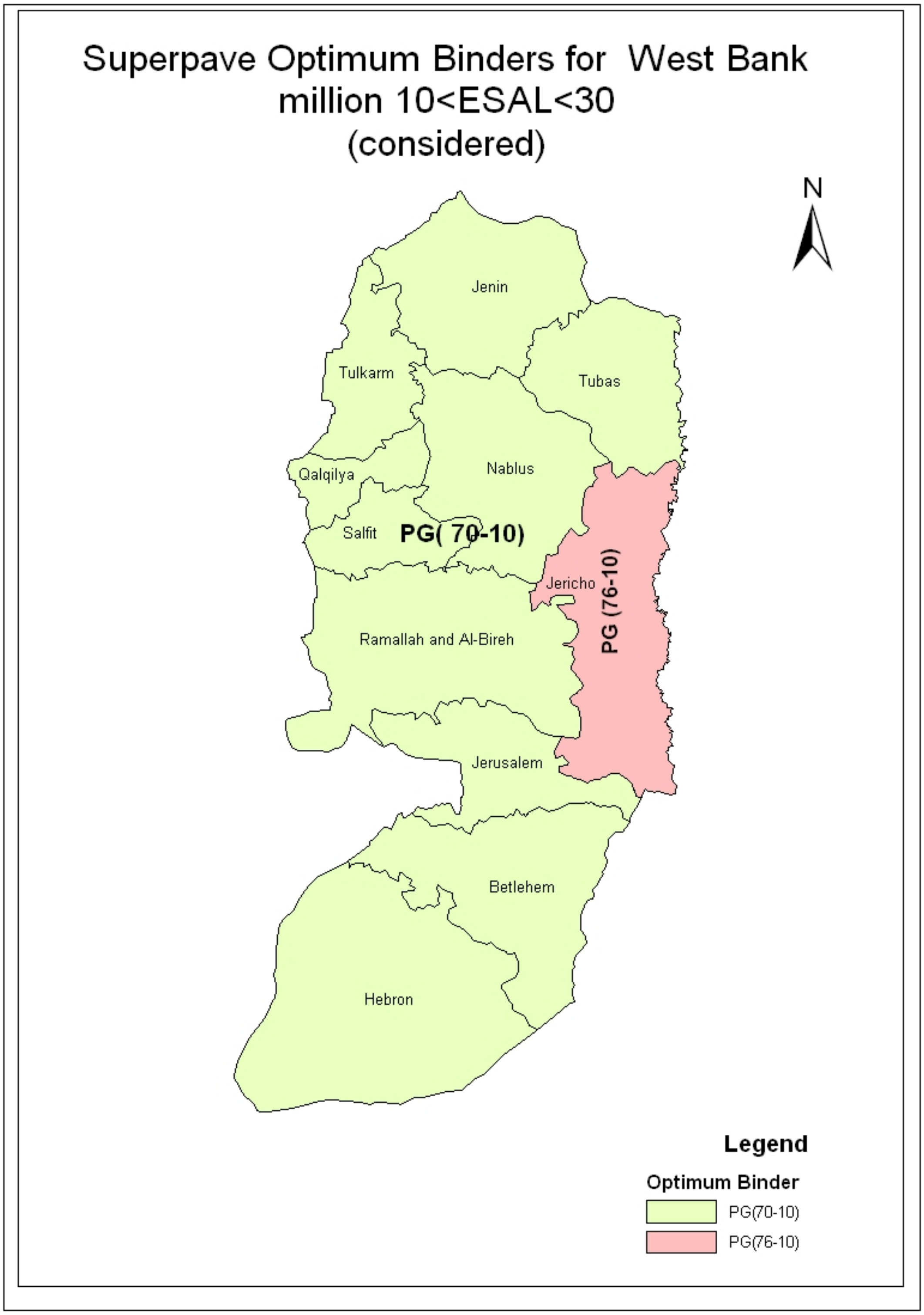

Figure (4.2): Optimum Binders For West Bank (10<ESAL< 30 million)

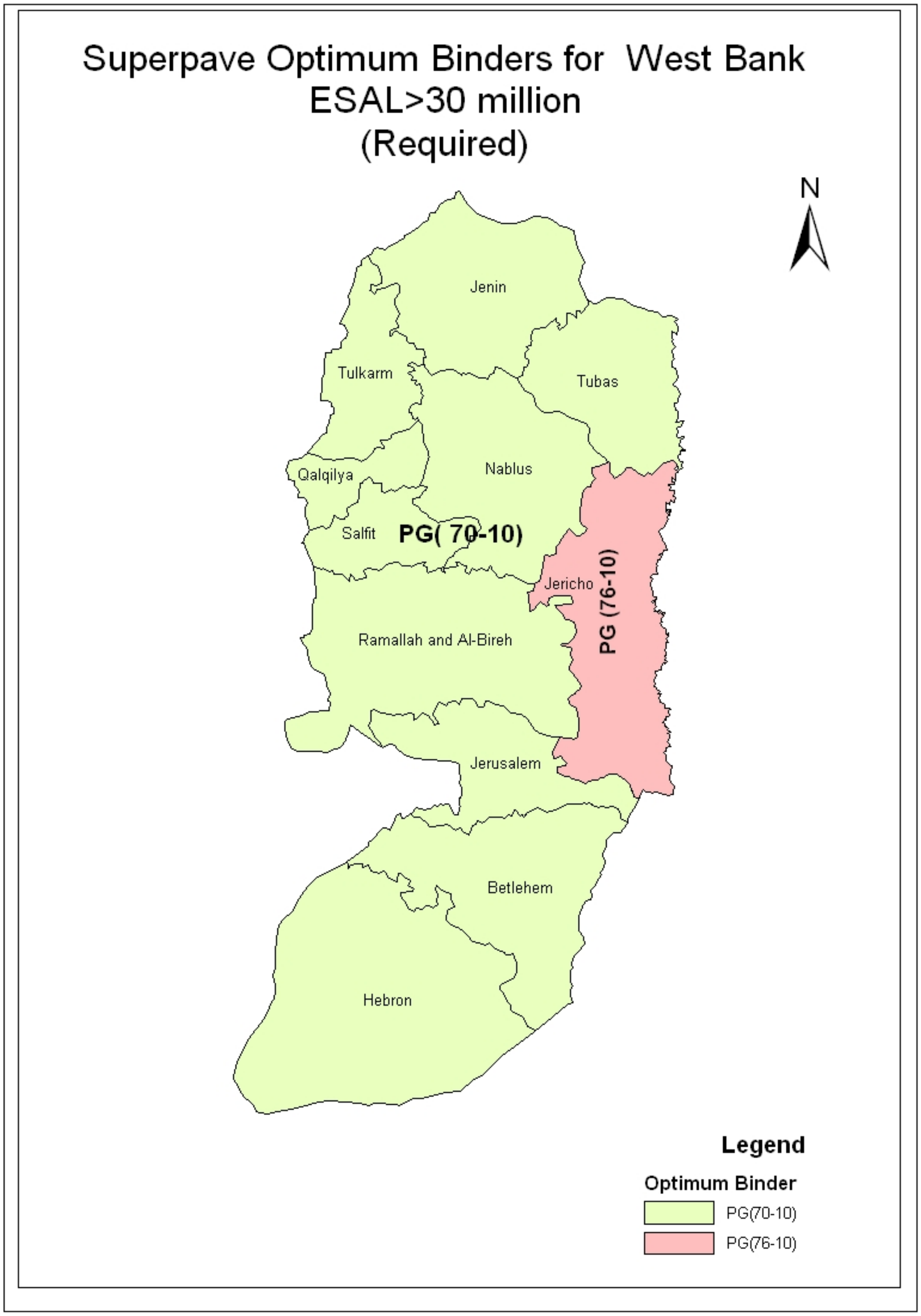

Figure (4.3): Optimum Binders For West Bank (ESAL>30 million)

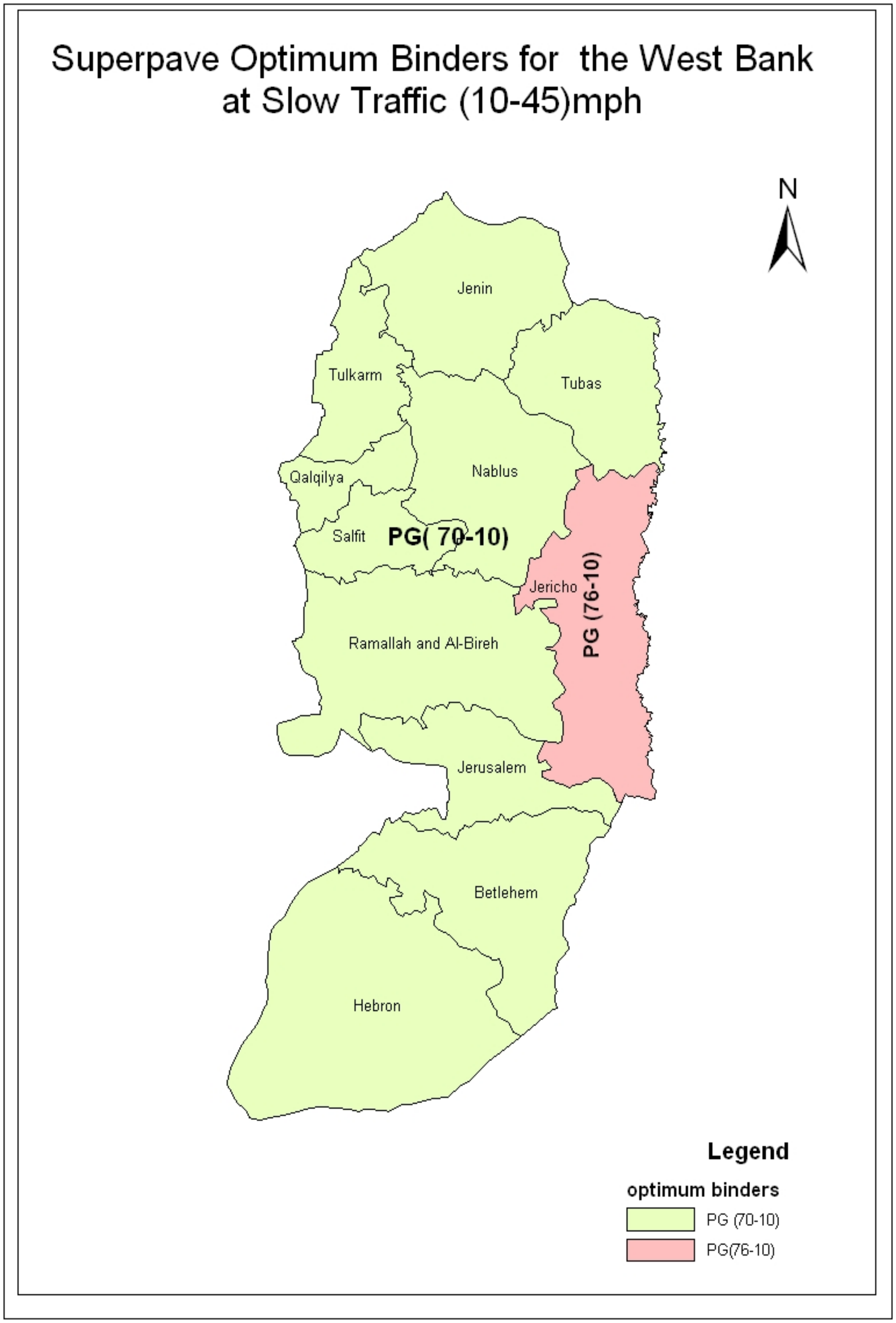

Figure (4.4): Optimum Binders For West Bank (slow traffic)

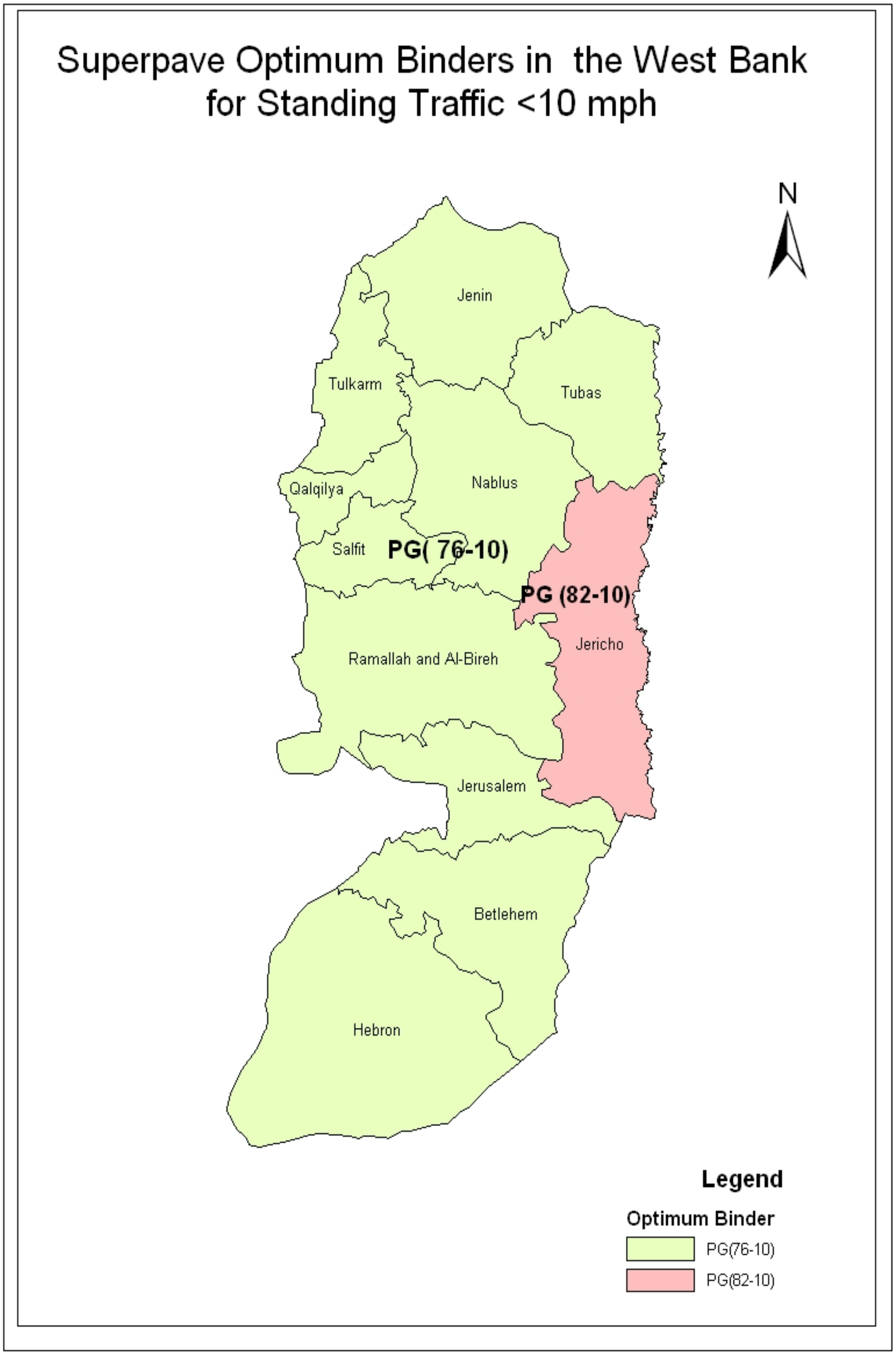

Figure (4.5): Optimum Binder in the West Bank (standing traffic)

4.6 Local Binder Used in the West Bank

According to the study of Asi (2005) about performance evaluation of Superpave and Marshall asphalt mix design to suite Jordan climate and traffic conditions, the study stated that the properties of the locally used binders in Jordan (60/70 penetration asphalt) achieve the specification of Superpave performance grade PG 64-16. As the same binder is used in the West Bank (60/70 penetration asphalt), it achieves the properties of PG 64-16 grade.

Based on Asi (2005), this asphalt has met both high temperature property requirements up to the high temperature 64C^o and to the low temperature physical properties requirements of at least -16C^o.

Since most regions in Palestine require a binder grade of PG 64-10, the local binder (60/70 penetration asphalt) can be used in most areas of West Bank excluding Jericho. In Jericho area, local asphalt should be modified to shift its grade to PG 70-10.

4.7 Comparison between Local Gradation and Superpave Gradation Specifications

Based on local practices in the Palestinian area, the following gradation is used for asphalt mixes.

Table (4.9): Local Gradation Specifications (3/4 inch maximum size)

Sieve Size		Control Points	
(mm)	(U.S.)	Lower	Upper
19	3/4 inch	100	-
12.5	1/2 inch	74	95
9.5	3/8 inch	60	86
4.75	No. 4	40	65
2.36	No. 8	-	-
1.18	No. 16	-	-
0.60	No. 30	-	-
0.30	No. 50	-	-
0.15	No. 100	-	-
0.075	No. 200	3	8

Source: The Palestinian Economic Council for Development and Reconstruction, 2008

However, the Superpave system uses different gradation, which is shown in Table 4.10.

Table (4.10): Superpave Gradation for 12.5 mm (1/2 inch) Nominal Size (3/4 inch maximum size)

Sieve Size		Control Points		Restricted Zone	
(mm)	(U.S.)	Lower	Upper	Lower	Upper
19	3/4 inch	100	-	-	-
12.5	1/2 inch	90	100	-	-
9.5	3/8 inch	-	90	-	-
4.75	No. 4	-	-	-	-
2.36	No. 8	28	58	39.1	39.1
1.18	No. 16	-	-	25.6	31.6
0.60	No. 30	-	-	19.1	23.1
0.30	No. 50	-	-	15.5	15.5
0.15	No. 100	-	-	-	-
0.075	No. 200	2	10	-	-

Note: Nominal maximum size is one sieve larger than the first sieve retains more than 10 percent of the soil

Source: Washington State DOT, 2007

There are major differences between the two gradation systems, as shown in Tables 4.9 and 4.10. The control points are completely different, and Superpave specifications use restricted zone gradation which is established between (0.3 mm) sieve and the sieve (2.36 mm).

For the soil blend to be acceptable, its gradation must not pass within the restricted zone. Soils that have gradation within a restricted zone have been found to create compaction problems during construction and tend to have inadequate voids in mineral aggregate (Garber and Hoel, 2002).

CHAPTER FIVE

CONCLUSIONS AND RECOMMENDATIONS

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5.1 Conclusions

Based on the results and analysis performed as apart of this thesis, the following can be concluded.

1- It was found that two asphalt grades are required for West Bank regions; PG 64-10 is suitable for most areas in the West Bank except Jericho region, which requires PG 70-10.

2- Jericho City is the most critical case in terms of the type of binder selection among Palestinian cities because of its hottest climate.

3- The selected binder grades have to be shifted up one or two grades for slow or standing loads.

4- High temperature grades have to be shifted up in case of extraordinary high numbers of repetitions (higher than 30 million ESAL) of heavy loads.

5- The effect of reliability on binder selection in the Palestinian regions is minimal. Therefore the reliability factor of 98% should be used as it is the most common factor.

6- There are obvious differences between local gradation and Superpave gradation requirements; therefore, aggregates' and mixes' properties are expected to be different.

7- In General, there were gaps in the available temperature data for 10 years as part of the Superpave requirements. Nonetheless, the available data was used because this was the only available data and the preliminary statistical analysis showed that the data is still reasonable.

5.2 Recommendations

1- As the Superpave mixes showed better performance over Marshall mixes, there should be shifting in the West Bank from the presently used Marshall mix design procedure to the Superpave mix design procedure.

2- In steep climbing lanes, parking of heavy trucks, or at intersections where traffic will be standing or there is a reduction in speed of heavy traffic, it is recommended to shift up local asphalt grade by two grades. The required grade is PG 76-10. To achieve this grade, a polymer should be used in modifying local asphalt.

3- Shifting by one grade due to the ESAL occurs at the principal arterials, which have high percentage of heavy traffic.

4- As the local binder (60/70 penetration) achieves the requirement of the Superpave binder; therefore, it is recommended to be used for the West Bank regions except for Jericho region, which requires PG70-10 binder grade.

5- Since the Superpave systems is recommended to be locally used, it is; therefore, recommended to adopt its aggregate gradation system and requirements instead of the local gradation system.

6- It is recommended to study Gaza Strip case as was done in the West Bank. This was not included in this thesis because of the unavailability of temperature data for Gaza Strip.

7- One of the limitations of this research is the unavailability of Superpave lab equipments in the Palestinian areas. Therefore, it is recommended to have Superpave laboratory in the West Bank to do further researches on this subject.

8- The related government agency (Department of Meteorology at the Ministry of Transportation) should continuously collect detailed temperature data for all the West Bank and Gaza regions since such data is needed for the Superpave system.

9- It is recommended to have efficient governmental monitoring on the local asphalt factories to achieve high quality of the Asphalt mixes.

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APPENDICES

Appendix A

GENERAL INFORMATION ABOUT ASPHALT MIXES AND SUPERPAVE METHOD

A.1 Origin of the Asphalt

Asphalt is defined by the American Society for Testing and Material (ASTM) as dark brown to black cementtitious material in which the predominating constituents are bitumens which occurs in nature or is obtained in petroleum processing." As cement, asphalt is especially valuable in construction because it is strong readily adhesive, highly waterproof, and durable. It provides limited flexibility to mixtures of mineral aggregate which it is usually combined. Although solid or semi solid at ordinary atmospheric temperatures, asphalt may be readily liquefied by applying heat, dissolving it in petroleum solvents, or by emulsifying it in water.

Man has been aware of the adhesive and water proofing properties of the asphalt. Surface accumulations of petroleum, forced upward by geological forces, leave behind naturally occurring lakes of asphalt which have been hardened after exposure to the weather elements. Current examples of these deposits include Trinidad lake asphalt on the island of Trinidad off the northern coast of Venezuela Figure A.1. Natural asphalt is also found impregnated within porous rock, such as sandstone or lime stone, called rock asphalt. Natural asphalts were used by the ancient Babylonians, Egyptians, Greek, and Romans as road building (Asphalt Institute, 2001).

Figure (A.1): Trinidad Lake Asphalt (source: Washington State DOT, 2007)

A.2 Petroleum Asphaltic Materials

The asphaltic material obtained from the distillation of petroleum is in the form of different types of asphalts. These include asphalt cements, slow-curing liquid asphalts, rapid-curing liquid asphalt, and asphalt emulsion (Garber and Hoel, 2002).

A.3 Refining Process

The refining process used to obtain petroleum asphalt is divided into two main groups, namely, fractional distillation and destructive distillation (cracking).

The fractional distillation involves the separation of the different material in the crude petroleum without significant changes in the chemical composition of each material. The destructive processes involve the application of high temperature and pressure.

A.3.1 Fractional Distillation

The fractional distillation process removes the different volatile materials in the crude oil at successively higher temperatures until the petroleum asphalt is obtain as residue. Steam or vacuum is used to gradually increase the temperature.

Steam distillation is continuous flow process in which the crude petroleum is pumped through tube stills or stored in batches, and the temperature is increased gradually to facilitate the evaporation of the different materials at different temperatures tube stills are more efficient than batches and are therefore preferred in modern refineries.

Figure A.2 is shows a flow chart for the interrelation of the different materials that can be obtained from the fractional distillation of crude petroleum.

After increasing the temperature of the crude in the tube still it is injected into a bubble tower, which consist of vertical cylinder into which are built several trays or platforms one above the other. The first separation of material occurs in this tower. The lighter fraction of the evaporated materials collect on the top tray, and the heavier fractions collect in successive trays, with the heaviest residue containing asphalt remaining at the bottom of the distillation tower. The products obtained during this first of separation are gasoline, kerosene, diesel fuel, lubricating oil and heavy residual material that contains the asphalt the various fractions collected are stored and further refined into specific grades of petroleum products. Note that a desired consistency of the residue can be obtained by continuing the distillation process. Attainment of the desired consistency is checked by measuring the temperature of the residue or by observing the character of the distillate. The residue becomes harder the longer the distillation process is continued.

It can seen from the Figure 2.2 the further processing of the heavy residue obtained after the first separation will give asphalt cement of different penetration grades-slow curing and rapid curing asphalts _ depending on the additional processing carried out (Garber and Hoel, 2002).

A.3.2 Destructive Distillation

Cracking are used when larger amount of the lights fractions of the materials such as motors fuels are required. Intense heat and high pressure are applied to product chemical changes in the material. Although several specific method of cracking exist, it is generally involves application of temperature as high as 1100°F and pressure higher than 735 ib/in^2. To obtain the desired effect, the asphaltic material obtained from cracking is not widely used in paving because it is more susceptible to weather changes than is that produced from fractional distillation (Garber and Hoel, 2002).

Asphalt binder specifications used to be relatively lenient, and gave refiners a high level of production flexibility. Therefore, refiners tended to view asphalt as a simple, convenient way to use the residual material from the refinery operation. Partially as a result of Superpave specifications asphalt binder specifications are now more stringent and asphalt refiners increasingly perceive asphalt as a value-added product. Superpave specifications have also caused many refiners to reevaluate their commitment to asphalt production; some have made a strategic decision to de-emphasize or cease asphalt production, though others have renewed their efforts to produce high-quality binders (Washington State DOT, 2007).

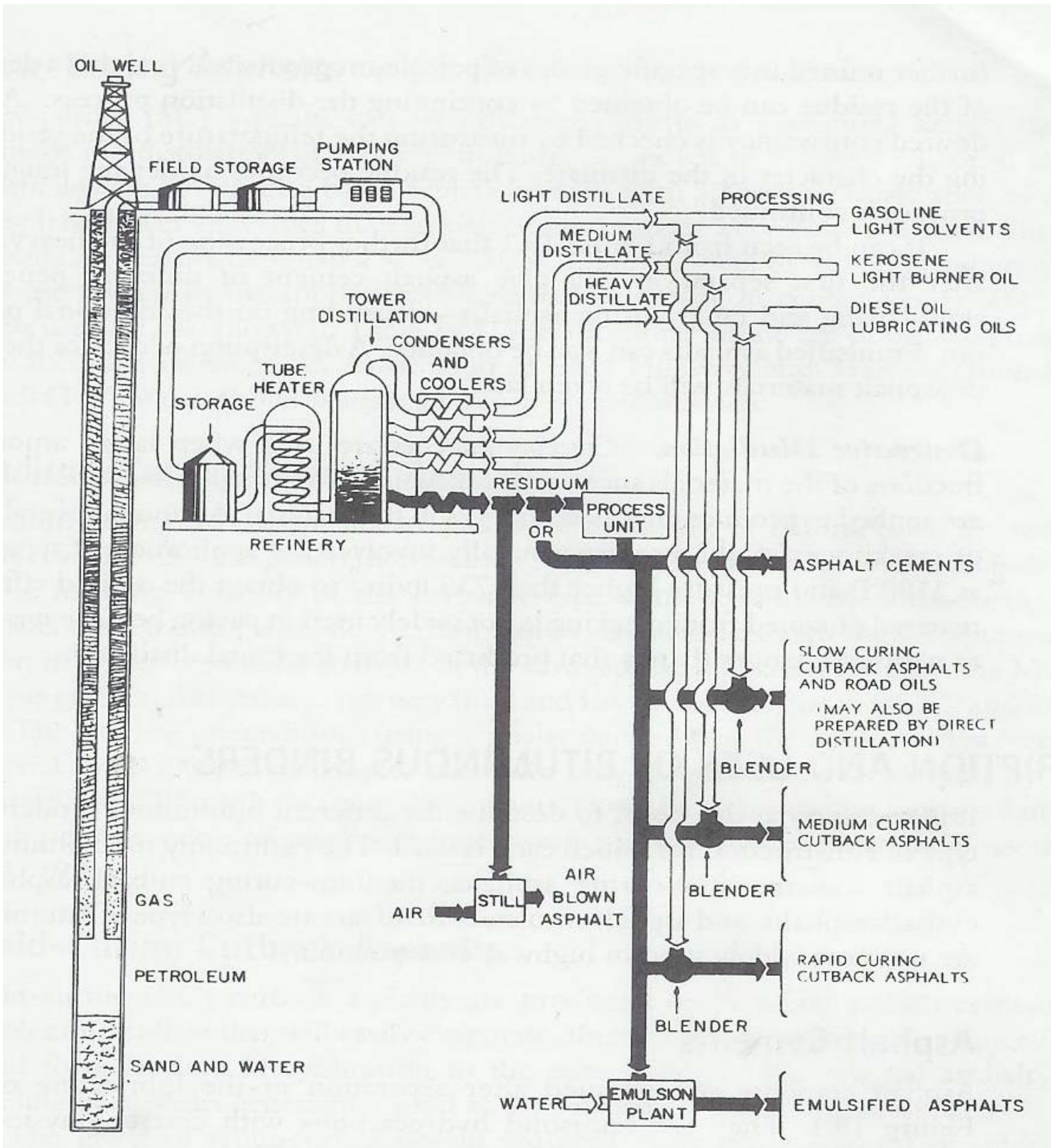

Figure (A.2): Fractional Distillation of Crude Petroleum

Source: (Garber and Hoel, 2002).

A.4 Asphalt Chemical Properties

Asphalt binders can be characterized by their chemical composition although they rarely are for HMA pavements. However, it is an asphalt binder's chemical properties that determine its physical properties. Therefore, a basic understanding of asphalt chemistry can help one understand how and why asphalt behaves the way it does. This subsection briefly describes the basic chemical composition of asphalts and why they behave as they do.

A.4.1 Basic Composition

Asphalt chemistry can be described on the molecular level as well as on the intermolecular (microstructure) level. On the molecular level, asphalt is a mixture of complex organic molecules that range in molecular weight from several hundred to several thousand. Although these molecules exhibit certain behavioral characteristics, the behavior of asphalt is generally ruled by behavioral characteristics at the intermolecular level – the asphalt's microstructure (Robertson, 1991).

The asphalt chemical microstructure model described here is based on SHRP findings on the microstructure of asphalt using nuclear magnetic resonance (NMR) and chromatography techniques. The SHRP findings describe asphalt microstructure as a dispersed polar fluid (DPF). The DPF model explains asphalt microstructure as a continuous three-dimensional association of polar molecules (generally referred to as "asphaltenes") dispersed in a fluid of non-polar or relatively low-polarity molecules (generally referred to as "maltenes"). All these molecules are capable of forming dipolar intermolecular bonds of varying strength. Since these intermolecular bonds are weaker than the bonds that hold the basic organic hydrocarbon constituents of asphalt together, they will break first and control the behavioral characteristics of asphalt. Therefore, asphalt's physical characteristics are a direct result of the forming, breaking and reforming of these intermolecular bonds or other properties associated with molecular superstructures.

The result of the above chemistry is a material that behaves (1) elastically through the effects of the polar molecule networks, and (2) viscously because the various parts of the polar molecule network can move relative to one another due to their dispersion in the fluid non-polar molecules (Washington State DOT, 2007).

A.4.2 Asphalt Behavior as a Function of its Chemical Constituents

Robertson et al. (1991) describe asphalt behavior in terms of its failure mechanisms. They describe each particular failure mechanism as a function of asphalt's basic molecular or intermolecular chemistry. This section is a summary of Robertson et al (1991).

> **Aging**. Some aging is reversible, some is not. Irreversible aging is generally associated with oxidation at the molecular level. This oxidation increases an asphalt's viscosity with age up until a point when the asphalt is able to quench (or halt) oxidation through immobilization of the most chemically reactive elements. Reversible aging is generally associated with the effects of molecular organization. Over time, the molecules within asphalt will slowly reorient themselves into a better packed, more bound system. This

results in a stiffer, more rigid material. This thixotropic aging can be reversed by heating and agitation.

Rutting and permanent deformation. If the molecular network is relatively simple and not interconnected, asphalt will tend to deform in elastically under load (e.g., not all the deformation is recoverable). Additionally, asphalts with higher percentages of non-polar dispersing molecules are better able to flow and plastically deform because the various polar molecule network pieces can more easily move relative to one another due to the greater percentage of fluid non-polar molecules.

Fatigue cracking. If the molecular network becomes too organized and rigid, asphalt will fracture rather than deform elastically under stress. Therefore, asphalts with higher percentages of polar, network-forming molecules may be more susceptible to fatigue cracking.

Thermal cracking. At lower temperatures even the normally fluid non-polar molecules begin to organize into a structured form. Combined with the already-structured polar molecules, this makes asphalt more rigid and likely to fracture rather than deform elastically under stress.

Stripping. Asphalt adheres to aggregate because the polar molecules within the asphalt are attracted to the polar molecules on the aggregate surface. Certain polar attractions are known to be disrupted by water (itself a polar molecule). Additionally, the polar molecules within asphalt will vary in their ability to adhere to any one particular type of aggregate.

Moisture damage. Since it is a polar molecule, water is readily accepted by the polar asphalt molecules. Water can cause stripping and/or can decrease asphalt viscosity. It typically acts like a solvent in asphalt and results in reduced strength and increased rutting. When taken to the extreme, this same property can be used to produce asphalt emulsions. Interestingly, from a chemical point-of-view water should have a greater effect on older asphalt. Oxidation causes aged (or older) asphalts to contain more polar molecules. The more polar molecules asphalt contains, the more readily it will accept water. However, the oxidation aging effects probably counteract any moisture-related aging effects.

In summary, asphalt is a complex chemical substance. Although basic chemical composition is important, it is asphalt's chemical microstructure that is most influential in its physical behavior. Although most basic asphalt binder failure mechanisms can be described chemically, currently there is not enough asphalt chemical knowledge to adequately predict performance. Therefore, physical properties and tests are used.

A.5 Physical Properties

Asphalt binders are most commonly characterized by their physical properties. An asphalt binder's physical properties directly describe how it will perform as a constituent in Hot Mix Asphalt (HMA) pavement. The challenge in physical property characterization is to develop physical tests that can satisfactorily characterize key asphalt binder parameters and how these parameters change throughout the life of an HMA pavement.

The earliest physical tests were empirically derived tests. Some of these tests (such as the penetration test) have been used for the better part of the 20th century with good results. Later tests (such as the viscosity tests) were first attempts at using fundamental engineering parameters to describe asphalt binder physical properties. Ties between tested parameters and field performance were still quite tenuous. Superpave binder tests, developed in the 1980s and 1990s, were developed with the goal of measuring specific asphalt binder physical properties that are directly related to field performance by engineering principles. These tests are generally a bit more complex but seem to accomplish a more thorough characterization of the tested asphalt binder.

This subsection, taken largely from Roberts et al. (1996), describes the more common U.S. asphalt binder physical tests. Asphalt binder tests specifically developed or adopted by the Superpave research effort are noted by a " Superpave" in their title. Sections that discuss Superpave tests also discuss relevant field performance information as well as the engineering principles used to develop the relationship between test and field performance.

A.5.1 Durability

Durability is a measure of how asphalt binder physical properties change with age (sometimes called age hardening). In general, as an asphalt binder ages, its viscosity increases and it becomes more stiff and brittle. Age hardening is a result of a number of factors, the principal ones being (Washington State DOT, 2007).

Oxidation: The reaction of oxygen with the asphalt binder.

Volatilization: The evaporation of the lighter constituents of asphalt binder. It is primarily a function of temperature and occurs principally during HMA production.

Polymerization: The combining of like molecules to form larger molecules. These larger molecules are thought to cause a progressive hardening.

Thixotropy; The property of asphalt binder whereby it "sets" when unagitated. Thixotropy is thought to result from hydrophilic suspended particles that form a lattice structure throughout the asphalt binder. This causes an increase in viscosity and thus, hardening (Exxon, 1997). Thixotropic effects can be somewhat reversed by heat and agitation. HMA pavements with little or no traffic are generally associated with thixotropic hardening. **Syneresis**: The separation of less viscous liquids from the more viscous asphalt binder molecular network. The liquid loss hardens the asphalt and is caused by shrinkage or rearrangement of the asphalt binder structure due to either physical or chemical changes. Syneresis is a form of bleeding (Washington State DOT, 2007).

Separation. The removal of the oily constituents, resins or asphaltenes from the asphalt binder by selective absorption of some porous aggregates.

There is no direct measure for asphalt binder aging. Rather, aging effects are accounted for by subjecting asphalt binder samples to simulated aging then conducting other standard physical tests (such as viscosity, dynamic shear rheometer (DSR), bending beam rheometer (BBR) and the direct tension test (DTT)). Simulating the effects of aging is important because an asphalt binder that possesses a certain set of properties in its as-supplied state, may possess a different set of properties after aging. Asphalt binder aging is usually split up into two categories:

Short-term aging. This occurs when asphalt binder is mixed with hot aggregates in an HMA mixing facility.

Long-term aging. This occurs after HMA pavement construction and is generally due to environmental exposure and loading (Washington State DOT, 2007).

Sarsam (2007) studied the properties of asphalt aging, it was concluded from this study that aging causes reduction in marshal stability and increase marshal flow. It also increase Haveem cohesion for law and moderate asphalt percentage in gap graded mixes. Also aging has significant negative effect on flexural strength.

A.6 Penetration Grading

The penetration grading system was developed in the early 1900s to characterize the consistency of semi-solid asphalts. Penetration grading quantifies the following asphalt concrete characteristics:

- Penetration depth of a 100 g needle 25° C (77° F)

- Flash point temperature

- Ductility at 25° C (77° F)

- Solubility in trichloroethylene

- Thin-film oven test (accounts for the effects of short-term aging)

- Retained penetration

- Ductility at 25° C (77° F)

Penetration grading basic assumption is that the less viscous the asphalt, the deeper the needle will penetrate. This penetration depth is empirically (albeit only roughly) correlated with asphalt binder performance. Therefore, asphalt binders with high penetration numbers (called "soft") are used for cold climates while asphalt binders with low penetration numbers (called "hard") are used for warm climates. Penetration grading key advantages and disadvantages are listed in Table A.1.

Penetration grades are listed as a range of penetration units (one penetration unit = 0.1 mm) such as 120 – 150. Penetration grades specified in AASHTO M 20 and ASTM D 946 are listed in Table A.2.

A few states still have provisions for the penetration grading system. These will most likely disappear as the Superpave PG system becomes more prevalent.

Table (A.1): Advantages and Disadvantages of the Penetration Grading

Advantages	Disadvantages
The test is done at 25° C (77° F), which is reasonably close to a typical pavement average temperature.	The test is empirical and does not measure any fundamental engineering parameter such as viscosity.
May also provide a better correlation with low-temperature asphalt binder properties than the viscosity test, which is performed at 60° C (140° F).	Shear rate is variable and high during the test. Since asphalt binders typically behave as a non-Newtonian fluid at 25° C (77° F), this will affect test results.
Temperature susceptibility (the change in asphalt binder rheology with temperature) can be determined by conducting the test at temperatures other than 25° C (77° F).	Temperature susceptibility (the change in asphalt binder rheology with temperature) cannot be determined by a single test at 25° C (77° F).
The test is quick and inexpensive. Therefore, it can easily be used in the field.	The test does not provide information with which to establish mixing and compaction temperatures.

Source: Washington State DOT, 2007

Table (A.2): AASHTO M 20 and ASTM D 946 Penetration Grades

Penetration Grade	Comments
40 – 50	Hardest grade.
60 – 70	Typical grades used in the U.S.
85 – 100	Typical grades used in the U.S.
120 – 150	
200 – 300	Softest grade. Used for cold climates such as northern Canada (Roberts et al., 1996)

Source: Washington State DOT, 2007

A.7 Viscosity Grading

In the early 1960s an improved asphalt grading system was developed that incorporated a rational scientific viscosity test. This scientific test replaced the empirical penetration test as the key asphalt binder characterization. Viscosity grading quantifies the following asphalt binder characteristics:

- Viscosity at 60° C (140° F)
- Viscosity at 135° C (275° F)
- Penetration depth of a 100 g needle applied for 5 seconds at 25° C (77° F)
- Flash point temperature
- Ductility at 25° C (77° F)
- Solubility in trichloroethylene
- Thin film oven test (accounts for the effects of short-term aging):
    - Viscosity at 60° C (140° F)
    - Ductility at 25° C (77° F)

Viscosity grading can be done on original (as-supplied) asphalt binder samples (called AC grading) or aged residue samples (called AR grading). The AR viscosity test is based on the viscosity of aged residue from the rolling thin film oven test. With AC grading, the asphalt binder is characterized by the properties it possesses before it undergoes the HMA manufacturing process. The AR grading system is an attempt to simulate asphalt binder properties after it undergoes a typical HMA. Manufacturing process and thus, it should be more representative of how asphalt binder behaves in HMA pavements (Washington State DOT, 2007). Table A.3 lists key advantages and disadvantages of the viscosity grading system.

Table (A.3): Advantages and Disadvantages of Viscosity Grading

Advantages		Disadvantages
Unlike penetration depth, viscosity is a fundamental engineering parameter.		The principal grading (done at 25° C (77° F)) may not accurately reflect low-temperature asphalt binder rheology.
Test temperatures correlate well with: • 25° C (77° F) – average pavement temp. • 60° C (140° F) – high pavement temp. • 135° C (275° F) – HMA mixing temp.		When using the AC grading system, thin film oven test residue viscosities can vary greatly with the same AC grade. Therefore, although asphalt binders are of the same AC grade they may behave differently after construction.
Temperature susceptibility (the change in asphalt binder rheology with temperature) can be somewhat determined because viscosity is measured at three different temperatures (penetration only is measured at 25° C (77° F)).		The testing is more expensive and takes longer than the penetration test.
Testing equipment and standards are widely available.		

Source: Washington State DOT, 2007

Viscosity is measured in poise (cm-g-s = dyne-second/cm2, named after Jean Louis Marie Poiseuille). The lower the number of poises, the lower the viscosity and thus the more easily a substance flows. Thus, AC-5 (viscosity is 500 ± 100 poise at 60° C (140° F)) is less viscous than AC-40 (viscosity is 4000 ± 800 poise at 60° C (140° F)). Table A.4 shows standard viscosity grades for the AC and AR grading systems from AASHTO M 226 and ASTM D 3381. Typical grades used for HMA paving in the U.S. are AC-10, AC-20, AC-30, AR-4000 and AR 8000.

Table A.4: AASHTO M 226 and ASTM D 3381 Viscosity Grades

Standard	Grading based on Original Asphalt (AC)						Grading based on Aged Residue (AR)				
AASHTO M 226	AC-2.5	AC-5	AC-10	AC-20	AC-30	AC-40	AR-10	AR-20	AR-40	AR-80	AR-160
ASTM D 3381	AC-2.5	AC-5	AC-10	AC-20	AC-30	AC-40	AR-1000	AR-2000	AR-4000	AR-8000	AR-16000

Source: Washington DOT, 2007

A.8 Superpave Method

A.8.1 History

Under the Strategic Highway Research Program (SHRP), an initiative was undertaken in the USA to improve materials selection, and mixture design by developing:

1. A new mix design method that accounts for traffic loading and environmental conditions.

2. A new method of asphalt binder evaluation.

3. New methods of mixture analysis.

When SHRP was completed in 1993 it introduced these three developments and called them the Superior Performing Asphalt Pavement System (Superpave). Although the new methods of mixture performance testing have not yet been established, the mix design method is well-established.

Although in common use throughout the U.S., the previous grading systems are somewhat limited in their ability to fully characterize asphalt binder for use in HMA pavements. Therefore, as part of the Superpave research effort, new binder tests and specifications were developed to more accurately and fully characterize asphalt binders for use in HMA pavements. These tests and specifications are specifically designed to address HMA pavement performance parameters such as rutting, fatigue cracking and thermal cracking.

Superpave performance grading (PG) is based on the idea that an HMA asphalt binder's properties should be related to the conditions under which it is used. For asphalt binders, this involves expected climatic conditions as well as aging considerations. Therefore, the PG system uses a common battery of tests (as the older penetration and viscosity grading systems do) but specifies that a particular asphalt binder must pass these tests at specific temperatures that are dependant upon the specific climatic conditions in the area of use. Therefore, a binder used in the Jericho city would have different properties than one used in the Ramallah city for example. This concept is not new – selection of penetration or viscosity graded asphalt binders follows the same logic – but the relationships between asphalt binder properties and conditions of use are more complete and more precise with the Superpave PG system. Information on how to select a PG asphalt binder for a specific condition is contained in Section 2.8.2.4, Asphalt binder evaluation (Washington State DOT, 2007).

The standard method for PG asphalt binder grading is: AASHTO PP6: Practice for Grading or Verifying the Performance Grade of an Asphalt Binder.

A.8.2 Superpave Mix Design Method

One of the principal results from the Strategic Highway Research Program (SHRP) was the Superpave mix design method. The Superpave mix design method was designed to replace the Hveem and Marshall methods. The volumetric analysis common to the Hveem and Marshall methods provides the basis for the Superpave mix design method. The Superpave system ties asphalt binder and aggregate selection into the mix design process, and considers traffic and climate as well. The compaction devices from the Hveem and Marshall procedures have been replaced by a gyratory compactor and the compaction effort in mix design is tied to expected traffic.

This section consists of a general outline of the actual Superpave mix design method. This outline emphasizes general concepts and rationale over specific procedures (Washington State DOT, 2007).

A.8.2.1 Procedure

The Superpave mix design method consists of 7 basic steps:

1. Aggregate selection
2. Asphalt binder selection

3. Sample preparation (including compaction)

4. Performance Tests

5. Density and voids calculations

6. Optimum asphalt binder content selection

7. Moisture susceptibility evaluation

A.8.2.1.1 Aggregate Selection

Superpave specifies aggregate in two ways. First, it places restrictions on aggregate gradation by means of broad control points. Second, it places "consensus requirements" on coarse and fine aggregate angularity, flat and elongated particles, and clay content. Other aggregate criteria, which the Asphalt Institute (2001) calls "source properties" (because they are considered to be source specific) as shown in the Table A.5 such as L.A. abrasion, soundness and water absorption are used in Superpave but since they were not modified by Superpave they are not discussed (Washington State DOT, 2007).

Table A.5 Superpave Aggregate Source Requirement

Property	Value
Los Angeles Abrasion (500 revolutions)	30% maximum
Degradation Factor	
Wearing Course	30 minimum
Non-Wearing Course	20 minimum

Source: Washington State DOT, 2007

A.8.2.1.2 Gradation and Size

Aggregate gradation influences such key HMA parameters as stiffness, stability, durability, permeability, workability, fatigue resistance, frictional resistance, and resistance to moisture damage (Roberts et al., 1996). Additionally, the maximum aggregate size can be influential in compaction and lift thickness determination.

A.8.2.1.3 Gradation Specifications

Superpave mix design specifies aggregate gradation control points, through which aggregate gradations must pass. These control points are very general and are a starting point for a job mix formula. Superpave uses 9.5 mm (0.375 inch), 12.5 mm (0.5 inch), 19.0 mm (0.75 inch), 25.0 mm (1 inch), and 37.5mm (1.5 inch) mixes (Washington State DOT 2007). Table A.6 shows typical Superpave aggregate specifications for 37.5 mm nominal aggregate sizes.

A.8.2.2 Aggregate Blending

It is rare to obtain a desired aggregate gradation from a single aggregate stockpile. Therefore, Superpave mix designs usually draw upon several different aggregate stockpiles and blend them together in a ratio that will produce an acceptable final blended gradation. It is quite common to find a Superpave mix design that uses 3 or 4 different aggregate stockpiles. Figure A.3 gives an example of the blend gradation.

Table (A.6): Typical Superpave Aggregate Specifications for 37.5 mm (1.5 inch) Nominal Size

Sieve Size		Control Points		Restricted Zone	
(mm)	(U.S.)	Lower	Upper	Lower	Upper
50	2 inch	100	-	-	-
37.5	1.5 inch	90	100	-	-
25	1 inch	-	90	-	-
19	3/4 inch	-	-	-	-
12.5	1/2 inch	-	-	-	-
9.5	3/8 inch	-	-	-	-
4.75	No. 4	-	-	34.7	34.7
2.36	No. 8	15	41	23.3	27.3
1.18	No. 16	-	-	15.5	21.5
0.60	No. 30	-	-	11.7	15.7
0.30	No. 50	-	-	10.0	10.0
0.15	No. 100	-	-	-	-
0.075	No. 200	0	6	-	-

Source: Washington State DOT, 2007.

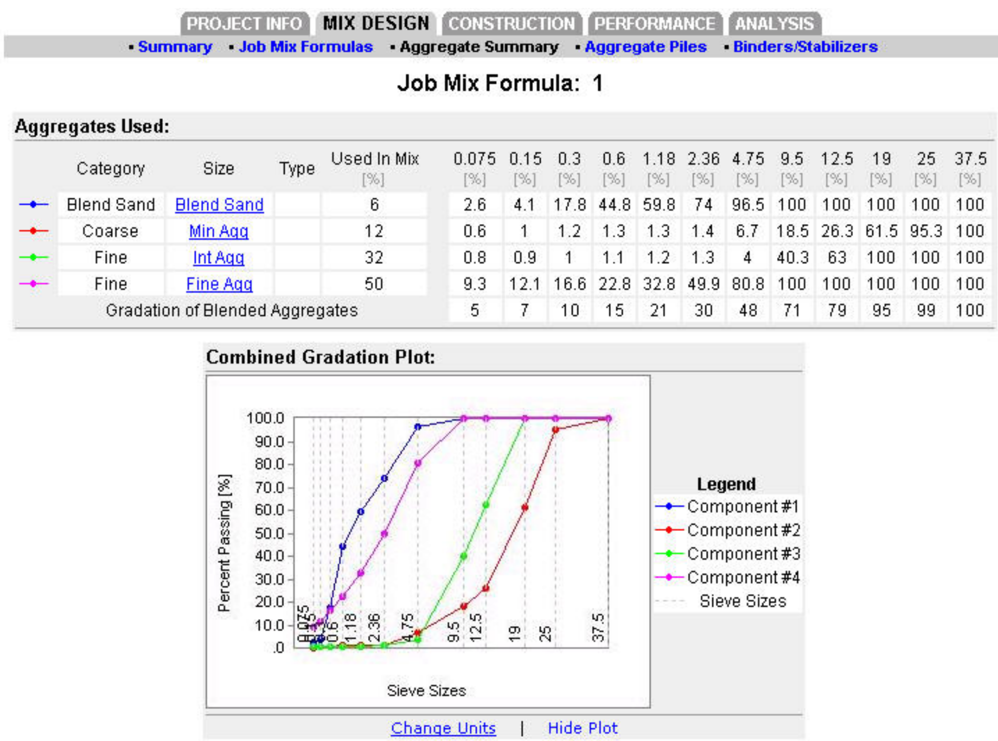

Figure A.3: Screen Shot from HMA View Showing a Typical Aggregate Blend from 4 Stockpiles

(Washington State DOT, 2007)

Typically, several aggregate blends are evaluated prior to performing a complete mix design. Evaluations are done by preparing an HMA sample of each blend at the estimated optimum asphalt binder content then compact it. Results from this evaluation can show whether or not a particular blend will meet minimum VMA.

A.8.2.2.1 Dust-to-Binder Ratio

In order to ensure the proper amount of material passing the 0.075 mm (No. 200) sieve (called "silt-clay" by AASHTO definition and "dust" by Superpave) in the mix, Superpave specifies a range of dust-to-binder ratio by mass. This can be done using equation A.1.

$$\text{Dust to binder ratio} = \frac{P_{0.075}}{P_{be}} \qquad \text{……………………………………Eqn A.1}$$

where: $P_{0.075}$ = Mass of particles passing the 0.075 mm (No. 200) sieve

P_{be} = Effective binder content or the total asphalt binder content of a paving mixture less the portion of asphalt binder that is lost by absorption into the aggregate particles.

Dust-to-binder ratio specifications are normally 0.6 - 1.2, but a ratio of up to 1.6 may be used at an agency's discretion (Washington State DOT, 2007)

A.8.2.3 Consensus Requirements

"Consensus requirements" came about because SHRP did not specifically address aggregate properties and it was thought that there needed to be some guidance associated with the Superpave mix design method. Therefore, an expert group was convened and they arrived at a consensus on several aggregate property requirements - the "consensus requirements". This group recommended minimum angularity, flat or elongated particle and clay content requirements based on:

- The anticipated traffic loading. Desired aggregate properties are different depending upon the amount of traffic loading. Traffic loading numbers are based on the anticipated traffic level on the design lane over a 20-year period regardless of actual roadway design life (AASHTO, 2000).

- Depth below the surface. Desired aggregate properties vary depending upon their intended use as it relates to depth below the pavement surface.

These requirements are imposed on the final aggregate blend and not the individual aggregate sources. Table A.7 shows Superpave Aggregate Consensus Requirement.

A.8.2.3.1 Coarse Aggregate Angularity

Coarse aggregate angularity is important to mix design because smooth, rounded aggregate particles do not provide minimum interlock with and will easily roll over one another allowing the movement within the hot mix pavement. This movement makes the HMA layer more susceptible to rutting . The Superpave design criteria recommend increasing the amount of fracture faces in the course aggregate with increasing traffic (Angelo,2008).

Fracture faces can be determined by any number of test procedure that are designed to determine percentage of the fracture faces Table A.8 Lists Superpave requirements for course aggregate angularity.

Table (A.7): Superpave Aggregate Consensus Requirement

Property	Value
Coarse Aggregate Angularity	
< 10 million ESALs	90/-*
≥ 10 million ESALs	-/90*
Fine Aggregate Angularity	45 minimum
Flat and Elongated Particles (5:1 ratio or greater)	10% maximum**
Clay Content (Sand Equivalent)	37% minimum
*The first number is a minimum requirement for one or more fractured faces and the second number is a minimum requirement for two or more fractured faces. **For > 0.3 million ESALs	

Source: Washington State DOT, 2007

Table A.8: Coarse Aggregate Angularity Requirements

20-yr Traffic Loading (in millions of ESALs)	Depth from Surface	
	< 100 mm (4 inches)	> 100 mm (4 inches)
< 0.3	55/-	-/-
0.3 to < 3	75/-	50/-
3 to < 10	85/80	60/-
10 to < 30	95/90	80/75
^{3} 30	100/100	100/100
Source: AASHTO, 2000. Note: The first number is a minimum requirement for one or more fractured faces and the second number is a minimum requirement for two or more fractured faces.		

A.8.2.3.2 Fine Aggregate Angularity

Fine aggregate Angularity is defined as the percent air voids present in loosely compacted aggregate smaller than 2.36, higher voids contents corresponds to higher fractured faces (Angelo, 1998).

Fine aggregate angularity is important to mix design for the same reasons as coarse aggregate angularity - rut prevention. Fine aggregate angularity is quantified by an indirect method often called the National Aggregate Association (NAA) flow test. This test consists of pouring the fine aggregate into the top end of a cylinder and determining the amount of voids. The more voids, the more angular the aggregate. Voids are determined by the equation 2.2.

$$Uncompacted\ Voids = \frac{V - \dfrac{W}{G_{sb}}}{V} \quad \text{……………………………Eqn..A.2}$$

where: V = volume of cylinder (ml).

W = weight of loose fine aggregate to fill the cylinder (g).

G_{sb} = bulk specific gravity of the fine aggregate.

Table A.9 shows the Superpave recommended fine aggregate angularity.

Table (A.9): Fine Aggregate Angularity Requirements

<table>
<tr><td rowspan="2">20-yr Traffic Loading
(in millions of ESALs)</td><td colspan="2">Depth from Surface</td></tr>
<tr><td>< 100 mm (4 inches)</td><td>> 100 mm (4 inches)</td></tr>
<tr><td>< 0.3</td><td>-</td><td>-</td></tr>
<tr><td>0.3 to < 3</td><td>40</td><td rowspan="3">40</td></tr>
<tr><td>3 to < 10</td><td rowspan="3">45</td></tr>
<tr><td>10 to < 30</td></tr>
<tr><td>≥ 30</td><td>45</td></tr>
<tr><td colspan="3">Source: AASHTO, 2000
Numbers shown represent the minimum un-compacted void content as a percentage of the total sample volume.</td></tr>
</table>

The standard test for fine aggregate angularity is AASHTO T 304: Un-compacted Void Content of Fine Aggregate.

A.8.2.3.3 Flat or Elongated Particles

An excessive amount of flat or elongated aggregate particles can be detrimental to HMA. Flat/elongated particles tend to breakdown during compaction (giving a different gradation than determined in mix design), decrease workability, and lie flat after compaction (resulting in a mixture with low VMA). Flat or elongated particles are typically identified using ASTM D 4791, Flat or Elongated Particles in Coarse Aggregate. Table A.10 shows the Superpave recommended flat or elongated particle requirements (Washington State DOT, 2007).

Table (A.10): Flat or Elongated Particle Requirements

20-yr Traffic Loading (in millions of ESALs)	Maximum Percentage of Particles with Length/Thickness > 5
< 0.3	-
0.3 to < 3	10
3 to < 10	
10 to < 30	
> 30	

Source: AASHTO, 2000.

A.8.2.3.4 Clay Content

The sand equivalent test measures the amount of clay content in an aggregate sample. If clay content is too high, clay could preferentially adhere to the aggregate over the asphalt binder. This leads to a poor aggregate-asphalt binder bonding and possible stripping. To prevent excessive clay content, Superpave uses the sand equivalent test requirements of Table A.11.

Table (A.11) : Sand Equivalent Requirements

20-yr Traffic Loading (in millions of ESALs)	**Minimum Sand Equivalent (%)**
< 0.3	40
0.3 to < 3	
3 to < 10	45
10 to < 30	
^{3} 30	50

Source: AASHTO, 2000.

APPENDIX B

TEMPERATURE DATA OBTAINED FROM DEPARTMENT OF METEOROLOGY

Table 1: Hottest Seven Days in Ramallah Each Year C^o

2000	2001	2002	2003	2004	2005	2006
37.5	35.4	39.4	33.6	35.6	NA	32.2
37.0	34.5	36.8	33.6	35.0	NA	30.1
34.2	34.4	36.5	33.5	33.2	NA	29.6
34.0	33.5	36.0	33.2	32.8	NA	29.5
34.0	32.2	35.6	33.0	32.8	NA	29.4
34.0	32.0	35.5	32.9	31.0	NA	29.4
33.6	31.8	35.0	32.8	31.0	NA	28.5
Average for maximum temperatures= 33.5 C^o						
standard deviation = 2.408						

Table 2: Minimum Temperatures in Ramallah C^o

2000	2001	2002	2003	2004	2005	2006
Min	**Min**	**Min**	**Min**	**Min**	**Min**	**Min**
-1.0	2.8	NA	0.0	0.0	NA	1.4
Average for minimum temperature= 0.64 C^o						
standard deviation = 1.48						

Table 3: Hottest Seven Days in Nablus Each Year C^o

98	99	2000	2001	2002	2003	2004	2005	2006
37.5	37.5	37.5	37.3	39.9	35.4	36.6	NA	35.6
37.0	37.5	37.0	37.0	38.4	34.5	36.2	NA	35.0
36.0	37.2	36.5	35.0	36.2	34.5	36.0	NA	34.5
36.0	36.0	36.0	34.8	34.6	34.0	36.0	NA	34.0
35.7	35.6	35.6	34.5	31.9	34.0	34.6	NA	34.0
35.4	35.5	35.2	34.2	31.8	33.8	34.4	NA	34.0
35.2	34.6	35.0	34.2	31.2	33.7	34.2	NA	33.6
Average for maximum temperatures = 35.3 C^o								
standard deviation = 1.71								

Table 4: Minimum Temperatures in Nablus Each Year C^o

98	99	2000	2001	2002	2003	2004	2005	2006
Min	**Min**	**Min**	**Min**	**Min**	**Min**	**Min**	**Min**	**Min**
0.0	3.4	0.0	3.8	NA	NA	0.8	NA	2.6
Average for minimum temperatures= 1.77 C^o								
standard deviation for minimum temperatures = 1.71								

Table 5: Hottest Seven Days in Tulkarm Each Year C^o

2007	2006	2005	2004	2003	2002	2001	2000	99	98
39.0	35.6	37	39.2	NA	NA	40.0	37.0	42.0	40.4
38.4	35.6	37	39.0	NA	NA	39.0	37.0	41.6	40.0
38.0	35.0	36.4	39.0	NA	NA	38.0	36.4	41.0	38.8
38.0	34.4	36.2	37.0	NA	NA	37.0	36.2	39.6	38.6
36.2	34.0	35.6	36.0	NA	NA	37.0	36.0	35.4	38.2
36.0	33.8	35.2	36	NA	NA	37.0	36.0	35.0	38.0
36.0	33.8	35.0	35.0	NA	NA	37.0	34.4	34.0	38.0
Average for maximum temperatures = 37.1 C^o									
standard deviation = 2									

Table 6: Minimum Temperatures in Tulkarm Each Year C^o

Min	Min	Min	Min	Min	Min	Min	Min	Min	Min
8.0	6.0	2.0	4.0	NA	NA	5.0	2.0	6.0	3.0
Average for minimum temperatures = 4.5 C^o									
standard deviation = 2.14									

Table 7: Hottest Seven Days in Jenin Each Year C^o

98	99	2000	2001	2002	2003	2004	2005	2006
40.0	41.2	39.4	41.4	NA	NA	39.4	NA	NA
39.4	41.0	38.8	40.6	NA	NA	39	NA	NA
39.4	40.4	38.6	39.0	NA	NA	38.6	NA	NA
38.4	40.0	38.4	38.4	NA	NA	37.8	NA	NA
38.2	39.0	38.0	37.4	NA	NA	36.8	NA	NA
38.0	38.4	37.6	37.2	NA	NA	36.6	NA	NA
38.0	37.4	36.8	37.0	NA	NA	36.2	NA	NA
Average for maximum temperatures = 38.6 C^o								
Standard deviation = 1.34 C^o								

Table 8: Minimum Temperatures in Jenin Each Year C^o

98	99	2000	2001	2002	2003	2004	2005	2006
Min	**Min**	**Min**	**Min**	**Min**	**Min**	**Min**	**Min**	**Min**
2.0	4.8	3.0	3.6	-1.6	NA	4.8	NA	NA
Average for minimum temperatures = 2.77 C^o								
standard deviation = 2.4 C^o								

Table 9: Hottest Seven Days in Hebron Each Year C^o

99	2000	2001	2002	2003	2004	2005	2006
34.0	37.0	33.8	39.0	33.5	NA	35.0	34.0
32.5	36.5	33.5	36.8	33.4	NA	33.6	31.6
32.5	34.0	33.4	35.5	33.0	NA	33.0	31.5
32.5	33.0	32.5	35.5	32.8	NA	33.0	31.0
32.0	33.0	32.2	34.5	32.6	NA	33.0	30.4
31.2	32.6	32.0	34.5	32.2	NA	32.6	30.0
31.0	32.5	32.0	33.5	32.0	NA	32.0	30.0
Average for maximum temperatures = 33.2 C^o							
standard deviation = 1.71							

Table 10: Minimum Temperatures in Hebron Each Year C^o

99	2000	2001	2002	2003	2004	2005	2006
Min	**Min**	**Min**	**Min**	**Min**	**Min**	**Min**	**Min**
1.4	**-2.5**	**1.2**	**-1.0**	**-1.0**	**-1.2**	**-1.0**	**0.0**
Average for minimum temperatures = -0.733 C^o							
standard deviation = 1.39							

Table 11: Hottest Seven Days Temperatures in Maythalon Each Year C^o

2007	2006	2005	2004	2003	2002	2001	2000	99	98
39.5	NA	37	39.0	NA	NA	39.5	39.8	40.0	40
39.0	NA	36.5	38.6	NA	NA	39.5	39.5	40.0	39.5
38.5	NA	36.5	38.0	NA	NA	39.5	38.5	39.5	38.5
38.2	NA	36.0	38.0	NA	NA	37.5	38.0	37.5	38.0
38.0	NA	36.0	37.5	NA	NA	36.5	37.5	37.5	37.0
37.5	NA	35	37	NA	NA	36.0	37.5	37.5	37.0
37.0	NA	34.2	37.0	NA	NA	36.0	37.0	36.5	37.0
Average of the maximum temperatures = 37.8 C^o									
standard deviation = 1.37									

Table 12: Minimum Temperature in Maythalon Each Year C^o									
2007	**2006**	**2005**	**2004**	**2003**	**2002**	**2001**	**2000**	**99**	**98**
Min	**Min**	**Min**	**Min**	**Min**	**Min**	**Min**	**Min**	**Min**	**Min**
1.8	NA	-1.5	-2.5	NA	NA	-2.0	-1.5	0.5	1.0
Average of the minimum temperatures = -0.6 C^o									
standard deviation = 1.67									

Table 13: Hottest Seven Days in Jericho Each Year

98	99	2000	2001	2002	2003	2004	2005	2006
46.8	45.6	45.0	45.4	48.0	43.0	44.4	NA	44.4
45.2	45.4	44.2	45.4	47.0	43.0	43.8	NA	43.8
44.0	43.8	44.0	44.8	46.8	42.6	43.6	NA	43.6
43.0	43.2	43.4	43.8	46.4	42.2	43.0	NA	43.0
42.6	43.0	43.4	43.0	45.8	42.2	43.0	NA	43.0
42.4	42.6	43.0	43.0	45.0	42.0	42.8	NA	42.8
42.2	42.2	42.5	42.4	44.6	42.0	42.4	NA	42.4
Average of the maximum temperatures = 43.8 C^o								
standard deviation = 1.45								

Minimum Temperature in Jericho Each Year

98	99	2000	2001	2002	2003	2004	2005	2006
Min	Min	Min	Min	Min	Min	Min	Min	Min
3.4	5.4	2.6	3.0	4.0	5.2	5.6	NA	5.1
Average of the minimum temperatures = 4.3 C^o								
Standard deviation = 1.18								

APPENDIX C

SUPERPAVE GRADATION SPECIFICATIONS

Table 1: Superpave Gradation for 25 mm (1 inch) Nominal Size

Sieve Size		Control Points		Restricted Zone	
(mm)	(U.S.)	Lower	Upper	Lower	Upper
37.5	1.5 inch	100	-	-	-
25	1 inch	90	100	-	-
19	3/4 inch	-	90	-	-
12.5	1/2 inch	-	-	-	-
9.5	3/8 inch	-	-	-	-
4.75	No. 4	-	-	39.5	39.5
2.36	No. 8	19	45	26.8	30.8
1.18	No. 16	-	-	18.1	24.1
0.60	No. 30	-	-	13.6	17.6
0.30	No. 50	-	-	11.4	11.4
0.15	No. 100	-	-	-	-
0.075	No. 200	1	7	-	-

Table 2: Superpave Gradation for 19 mm (3/4 inch) Nominal Size

Sieve Size		Control Points		Restricted Zone	
(mm)	(U.S.)	Lower	Upper	Lower	Upper
25	1 inch	100	-	-	-
19	3/4 inch	90	100	-	-
12.5	1/2 inch	-	90	-	-
9.5	3/8 inch	-	-	-	-
4.75	No. 4	-	-	-	-
2.36	No. 8	23	49	34.6	34.6
1.18	No. 16	-	-	22.3	28.3
0.60	No. 30	-	-	16.7	20.7
0.30	No. 50	-	-	13.7	13.7
0.15	No. 100	-	-	-	-
0.075	No. 200	2	8	-	-

Source: Washington State DOT, 2007.

Table 3: Superpave Gradation for 12.5 mm (1/2 inch) Nominal Size

Sieve Size		Control Points		Restricted Zone	
(mm)	**(U.S.)**	**Lower**	**Upper**	**Lower**	**Upper**
19	3/4 inch	100	-	-	-
12.5	1/2 inch	90	100	-	-
9.5	3/8 inch	-	90	-	-
4.75	No. 4	-	-	-	-
2.36	No. 8	28	58	39.1	39.1
1.18	No. 16	-	-	25.6	31.6
0.60	No. 30	-	-	19.1	23.1
0.30	No. 50	-	-	15.5	15.5
0.15	No. 100	-	-	-	-
0.075	No. 200	2	10	-	-

Table 4: Superpave Gradation for 9.5 mm (3/8 inch) Nominal Size

Sieve Size		Control Points		Restricted Zone	
(mm)	**(U.S.)**	**Lower**	**Upper**	**Lower**	**Upper**
12.5	1/2 inch	100		-	-
9.5	3/8 inch	90	100	-	-
4.75	No. 4	-	90	-	-
2.36	No. 8	32	67	47.2	47.2
1.18	No. 16	-	-	31.6	37.6
0.60	No. 30	-	-	23.5	27.5
0.30	No. 50	-	-	18.7	18.7
0.15	No. 100	-	-	-	-
0.075	No. 200	2	10	-	-

Source: Washington State DOT, 2007.

جامعة النجاح الوطنية
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علاء شفيق لطفي عبد الله

إشراف
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د. خالد الساحلي

قدمت هذه الأطروحة استكمالا لمتطلبات درجة الماجستير في هندسة الطرق والمواصلات بكلية الدراسات العليا في جامعة النجاح الوطنية في نابلس، فلسطين.

2008

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الملخص

تهدف هذه الأطروحة بشكل عام إلى تطبيق نظام الرصفات الإسفلتية عالية الأداء (السوبر باف)، والذي تم تطويره في الولايات المتحدة الأمريكية ما بين عام 1987-1993 على المناطق الفلسطينية في الضفة الغربية. لقد تم الحصول على بيانات درجات الحرارة من دائرة الأرصاد الجوية، كما تم الحصول على مواقع المدن الفلسطينية (خط العرض بالدرجات) وذلك من الخرائط الجغرافية.

بعد تحليل البيانات تبين أن معظم المناطق الفلسطينية بحاجة إلى نوع واحد من اللواصق وهو PG 64-10، باستثناء منطقة أريحا والتي أظهرت النتائج أنها بحاجة إلى نوع لاصق PG 70-10.

وبالاستئناس بدراسات سابقة على اللاصق المستخدم في الأردن والذي يستخدم مثيله في المناطق الفلسطينية، تبين أنه يشابه خصائص اللاصق PG(64-16)، لذلك فإنه من الممكن استخدام اللاصق المحلي في جميع المناطق الفلسطينية باستثناء منطقة أريحا.

كذلك فقد جرى دراسة عدة حالات خاصة وذلك في المناطق التي يكون فيها حركة سير بطيئة أو ثابتة (مواقف حافلات)، وكذلك عندما يكون حجم السير كبير، وقد تم تحديد اللاصق المناسب لكل حالة حسب معايير نظام السوبرباف.

أخيرا فإنه يوصى باستخدام نظام السوبرباف في الأراضي الفلسطينية بما فيه من اختيار اللاصق ونسب الخلط التصميمية، حيث أن جميع الدراسات السابقة أظهرت وجود أفضلية لاستخدام هذا النظام مقارنة بنظام المارشال المستخدم حالياً.