Analytical and Experimental Study of Field Compaction of Asphalt Mixes

By

Khaled Anwar Kandil, B.Sc., M.Sc.

A thesis submitted to the
Faculty of Graduate Studies and Research
in partial fulfillment of the requirements for the degree of

Doctor of Philosophy

Department of Civil and Environmental Engineering
Carleton University
Ottawa, Canada
June 2002

The Doctor of Philosophy in Civil Engineering is a joint program with the University of
Ottawa, administrated by the Ottawa-Carleton Institute for Civil Engineering

© 2002

Khaled Anwar Kandil

National Library
of Canada

Acquisitions and
Bibliographic Services

395 Wellington Street
Ottawa ON K1A 0N4
Canada

Bibliothèque nationale
du Canada

Acquisitions et
services bibliographiques

395, rue Wellington
Ottawa ON K1A 0N4
Canada

Your file Votre référence

Our file Notre référence

The author has granted a non-exclusive licence allowing the National Library of Canada to reproduce, loan, distribute or sell copies of this thesis in microform, paper or electronic formats.

The author retains ownership of the copyright in this thesis. Neither the thesis nor substantial extracts from it may be printed or otherwise reproduced without the author's permission.

L'auteur a accordé une licence non exclusive permettant à la Bibliothèque nationale du Canada de reproduire, prêter, distribuer ou vendre des copies de cette thèse sous la forme de microfiche/film, de reproduction sur papier ou sur format électronique.

L'auteur conserve la propriété du droit d'auteur qui protège cette thèse. Ni la thèse ni des extraits substantiels de celle-ci ne doivent être imprimés ou autrement reproduits sans son autorisation.

0-612-79431-8

Canada

The undersigned recommend to the
Faculty of Graduate Studies and Research
acceptance of this thesis

Analytical and Experimental Study of Field
Compaction of Asphalt Mixes

submitted by:

Khaled Anwar Kandil, B.Sc., M.Sc.

in partial fulfilment of the requirements for
the degree of Doctor of Philosophy

Prof. A.O. Abd El Halim, Thesis Co-Supervisor
Department of Civil and Environmental Engineering
Carleton University

Prof. Said Easa, Thesis Co-Supervisor
Department of Civil Engineering
Ryerson University

Prof. Wayne Parker, Chairman
Department of Civil and Environmental Engineering
Carleton University

Prof. Nouman Ali, External Examiner
Department of Civil Engineering
Dalhousie University

Ottawa-Carleton Institute for Civil Engineering
June 2002

ABSTRACT

Compaction of asphalt concrete mixtures is a very complex process. A wide range of factors affect this process. Considerable research work has been carried out to investigate the effects of these factors on the compaction quality. Although the results of these studies have led to improving the quality of the compacted asphalt layers, the information gained can be regarded as general guidelines. Most of the obtained information is based on studies that were carried out to investigate the effect of specific factor(s) on the degree of compaction under a given set of conditions, that may not be encountered in many practical projects. Thus, an accurate modelling of the compaction process would greatly enhance the compaction process and lead to improved final product.

The research work presented in this thesis started with identifying the limitations associated with field compaction of asphalt mixes and establishing the need to develop a scientific framework for asphalt compaction in the field. The next step aimed at understanding the mix behaviour during compaction through a review of different behaviour of the engineering materials and the ingredient characterises of the asphalt mix. The following step focused on the design of a simple test procedure to determine the studied behaviour. Subsequently, a commercial paving project was selected to investigate the important compaction variables under actual field conditions. Thus, the mix behaviour was evaluated and compared under both field and laboratory conditions.

The results of both laboratory and field-testing were used to develop two compaction models. The developed models considered the significant variables affecting the compaction. The models can be used to evaluate the compaction quality as a function

of one or a combination of the input variables. The first model was developed based on correlating the results obtained from the laboratory testing to those monitored in the field. The second model involved using an existing computer software that allows using the output results of the suggested test procedure to represent the characteristics of the asphalt mixes under compaction conditions. This model has the extra advantage of modelling the asphalt multi-layer structure, including the effect of the underlying supporting layers.

ACKNOWLEDGEMENTS

The author would like to express his sincere appreciation to Professors A.O. Abd El Halim, Carleton University, and Said Easa, Ryerson University, for their supervision, guidance, suggestions, and continuous support throughout the course of this study. Special thanks are due to Professor Hassan, Carleton University, for his advice and assistance.

Appreciation is accorded to Katherine Lauter, Pavement Engineer, City of Ottawa for facilitating the field monitoring process; Staff of Highway Laboratory, City of Ottawa for their assistance in the field measurements; Dr. El Hussein Mohamed, Dr. Walaa Khogali and Solomon Tesfarmariam, Institute for Research in Construction (IRC), National Research Council, for giving the opportunity to use IRC equipment during the laboratory testing; Staff of Civil Engineering Laboratory, Carleton University, for their assistance in the laboratory investigation; Mr. Aly Sherif, Transport Canada, for his assistance. The financial support of NSERC, Carleton University, the Transportation Association of Canada (TAC), Fonds pour la formation de Chercheurs et l'Aide à la Recherche (FCAR), and Ontario Graduate Scholarship program (OGS) is gratefully acknowledge.

Finally, I would like to thank my father, wife, and sons for their endless help and support during the tough period of my study.

TABLE OF CONTENTS

ABSTRACT.....	iii
ACKNOWLEDGEMENTS.	v
TABLE OF CONTENTS.	vi
LIST OF TABLES.....	xi
LIST OF FIGURES.....	xiv
LIST OF APPENDICES	xix
CHAPTER 1. INTRODUCTION.....	1
1.1 General.....	1
1.2 Long-Term Performance of Asphalt Concrete Pavement.....	2
1.3 Importance of Asphalt Concrete Compaction.....	5
1.4 Problem Definition.....	6
1.5 Research Objectives	8
1.6. Thesis Organization.....	8
CHAPTER 2. ASPHALT CONCRETE COMPACTION.....	11
2.1 Introduction.....	11
2.2 Objectives and Importance of Asphalt Concrete Compaction.....	11
2.2.1 Stability.....	12
2.2.2 Fatigue Strength.....	16
2.2.3 Tensile and Shear Strengths.....	16
2.2.4 Durability.....	19
2.2.5 Permeability	21

2.3 Compaction Technology for Asphalt Concrete Compaction.....	23
2.3.1. Conventional Technology.....	23
2.3.2 New Technology.....	35
2.4 Factors Affecting Compaction Process.....	41
2.4.1 Material Properties.....	43
2.4.2 Environmental Variables.....	48
2.4.3 Laydown Site Conditions.....	53
2.4.4 Compaction Equipment.....	54
2.5 Summary.....	55
CHAPTER 3. RESEARCH PLAN AND ANALYTICAL INVESTIGATION.....	57
3.1 Introduction.....	57
3.2 Research Plan.....	57
3.3 Consolidation Theory for Fine-Grained Soil.....	62
3.3.1 Consolidation Process.....	62
3.3.2 Consolidation Theory: one-dimensional method.....	65
3.4 Description of Asphalt Concrete Compaction Using Consolidation Theory	69
3.5 Differences Between the Consolidation Theory and Compaction Process.....	72
3.6 Behaviour of Engineering Materials.....	73
3.6.1 Elastic and Plastic Strains.....	73
3.6.2 Creep Strain.....	78
3.7. Behaviour of Asphalt Concrete Mix.....	80

3.8 Viscous Behaviour for Asphalt Concrete Mixes.....	81
3.9 Behaviour of Asphalt Concrete Mix during Compaction.....	82
CHAPTER 4. FIELD COMPACTION AND DATA COLLECTION.....	85
4.1 Introduction.....	85
4.2 Project Description.....	87
4.2.1 Pavement Cross-section.....	87
4.2.2. Construction Process.....	90
4.2.3. Compactors Characteristics.....	100
4.2.4. Data and Measurements.....	101
4.3 Summary.....	112
CHAPTER 5. EXPERIMENTAL INVESTIGATION.....	113
5.1 Introduction.....	113
5.2 Development of a Test Procedure to Investigate the Behaviour of Asphalt Concrete Mixes during Compaction.....	113
5.2.1. Test Equipment.....	114
5.2.2 Specimens Preparation.....	117
5.2.3 Density versus Number of Blows.....	120
5.2.4 Test Variables.....	122
5.2.5 Pilot Testing.....	124
5.2.6 Achieving and Maintaining Test Temperature.....	132
5.3 Experimental Program.....	133
5.3.1 Elastic Strain.....	133
5.3.2 Permanent Strain.....	137

5.4 SUMMARY.....	139
CHAPTER 6 MODELLING THE COMPACTION PROCESS BASED ON	
EXPERIMENTAL AND FIELD RESULTS (Model I).....	140
6.1 Introduction.....	140
6.2 Regression Analysis.....	143
6.3 Modelling Elastic Behaviour of Asphalt Mix.....	143
6.4 Permanent Deformation of Asphalt Mix.....	145
6.5 Modelling of Asphalt Concrete Compaction.....	149
6.5.1 Compaction Models.....	149
6.5.2 Correlating Lab Results to Field Results.....	150
6.6 Summary.....	158
CHAPTER 7 MODELLING THE COMPACTION PROCESS USING	
EXISTING SOFTWARE (Model II).....	159
7.1. Introduction.....	159
7.2 Computer Software.....	159
7.3 Defining Supporting Layer Characteristics.....	161
7.3.1 Resilient Modulus of Subgrade.....	161
7.3.2 Resilient Modulus of Granular Layers.....	162
7.3.3 Asphalt Layers Characteristics.....	169
7.4 Compaction Model.....	174
7.4.1 Sensitivity Analysis.....	174
7.4.2 Compaction Modelling.....	175
7.5 Summary.....	181

CHAPTER 8. SUMMARY, FINDINGS AND RECOMMENDATIONS.....	182
8.1 Summary.....	182
8.2 Findings.....	184
8.3 Recommendations	187
REFERENCES.....	189
APPENDICES.....	196

LIST OF TABLES

Table 2.1 Influence of Degree of Compaction on Pavement Thickness Requirements.....	15
Table 2.2 Main Differences between Steel Roller and AMIR.....	39
Table 2.4 Factors Affecting Compaction Process	42
Table 4.1 Aggregate Gradation Specification Limits for Granular Layers.....	89
Table 4.2. Mix Properties for HL-8 and HL-3.....	89
Table 4.3 Specification Limits of HL-8 and HL-3.....	90
Table 4.4 Subgrade Characteristics.....	91
Table 4.5 Aggregate Gradation for Granular Layers.....	92
Table 4.6 Mix Properties For HL-8 and HL-3.....	92
Table 4.7 Aggregates Gradation and Binder Contents for Mixes HL-8 and HL-3.....	93
Table 4.8 Characteristics of Steel Roller used in the First Compaction Stage.....	100
Table 4.9 Characteristics of Pneumatic Roller used in the Second Compaction Stage.....	100
Table 4.10 Measured Field Density versus Number of Passes.....	104
Table 4.11 Gauge Reading versus Core Density for HL-3 and HL-8.....	105
Table 4.12 Corrected Field Density versus Number of Passes.....	106
Table 4.13 Temperature during Compaction Process.....	109
Table 4.14 Contact Time, Curve, and Pressure during Compaction of HL-8 (1st layer).....	110

Table 4.15 Contact Time, Curve, and Pressure during Compaction of HL-8 (2nd layer)	110
Table 4.16 Contact Time, Curve, and Pressure during Compaction of HL-3.....	111
Table 4.17 Water Content and Density of Granular layers.....	112
Table 5.1 Density versus Number of Blows for HL-8.....	120
Table 5.2 Density versus Number of Blows for HL-3.....	120
Table 5.3 Test Variables.....	123
Table 5.4 Irrecoverable Strain for Different Confining Media.....	128
Table 5.5 Density after Creep Test for Different Confining Media.....	131
Table 5.6 Test Conditions of the Experimental Program “case of elastic strain”...	134
Table 5.7 Results of Elastic Strain for Mix HL-8.....	135
Table 5.7 (continued) Results of Elastic strain for Mix HL-8.....	136
Table 5.8 Results of Elastic Strain for Mix HL-3.....	136
Table 5.8 (continued) Results of Elastic Strain for Mix HL-3.....	137
Table 5.9 Test Conditions of the Experimental Program “case of permanent strain”.....	138
Table 6.1 Field Densities for HL-8 and HL-3 used for Model Calibration.....	153
Table 6.2 Calculated Densities for HL-8 and HL-3.....	154
Table 6.3 Field versus Corrected Densities for First Layer of HL-8.....	156
Table 6.4 Field versus Corrected Densities for Second Layer of HL-8.....	157
Table 6.5 Field versus Corrected Densities for the Layer of HL-3.....	157
Table 7.1 Load Sequences used in the Resilient Modulus test.....	164
Table 7.2 Resilient Modulus Result versus Water Content for Granular A.....	164

Table 7.3 Granular Layers Characteristics.....	168
Table 7.4 Vertical Deformation versus Radius of Contact Radius.....	175
Table 7.5 Software Calculated Density versus Field Density for First Layer of HL-8.....	176
Table 7.6 Software Calculated Density versus Field Density for Second Layer of HL-8.....	177
Table 7.7 Software Calculated Density versus Field Density for Layer of HL-3.....	177
Table 7.8 Corrected Density versus Field Density for First Layer of HL-8.....	179
Table 7.9 Corrected Density versus Field Density for Second Layer of HL-8.....	180
Table 7.10 Corrected Density versus Field Density for the Layer of HL-3.....	180

LIST OF FIGURES

Figure 1.1 Typical Flexible Pavement Cross-section.....	2
Figure 1.2 Pavement Life Cycle.....	4
Figure 1.3 Research Plan.....	9
Figure 2.1. Marshall Test Machine.....	14
Figure 2.2. Effect of Relative Density on Required Design Thickness.....	15
Figure 2.3. Tensile and Compressive Strength in Flexible Pavement.....	17
Figure 2.4 Shear Testing Machine.....	18
Figure 2.5 Testing Pavement Surface by CISST/InSiSST.....	19
Figure 2.6 Effect of Initial Pavement Air Voids on Change in Binder Penetration.	20
Figure 2.7 Permeability – Relative Compaction relation for Ten Different Projects	22
Figure 2.8 Horse-Drawn Roller.....	24
Figure 2.9 Gellerat & Co. Roller.....	25
Figure 2.10 Morland & Co. Roller.....	25
Figure 2.11 Wittingham & Douglas Roller.....	25
Figure 2.12 First Success Steamroller	26
Figure 2.13 Different Shapes of Steamroller Manufactured Between 1904 –1934.	27
Figure 2.14 First Motor Roller.....	28
Figure 2.15 Different Shapes of Motor Rollers Manufactured Between 1904 – 1934.....	29
Figure 2.16 First Tandem Roller in U.S.....	29
Figure 2.17 First Gasoline Tandem Roller in U.S.....	30

Figure 2.18 Pneumatic Roller.....	31
Figure 2.19 Vibratory Roller.....	31
Figure 2.20 Recent Steel Roller.....	34
Figure 2.21 Recent Pneumatic Roller.....	34
Figure 2.22 Steel Roller and Pavement System.....	36
Figure 2.23 AMIR and Pavement System.....	36
Figure 2.24 Steel Roller “Small Scale Model”.....	37
Figure 2.25 Asphalt-Multi Integrated Roller AMIR “Small Scale Model”.....	37
Figure 2.26 Asphalt Multi-Integrated Roller (AMIR).....	38
Figure 2.27 Hot Iron Process Asphalt Compaction (HIPAC).....	40
Figure 2.28 Mix Shaving.....	43
Figure 2.29 Effect of Asphalt Viscosity on Compaction of Asphalt Concrete Mixes	45
Figure 2.30 Effect of Mixing Temperature on Percent Penetration Retained.....	45
Figure 2.31 Effect of Filler to Asphalt Ratio on Percent Theoretical Maximum Density	47
Figure 2.32 Mat Thickness versus Time for Mat to Cool to 80 °C (Spreading Mix Temperatures 150, 120 °C).....	49
Figure 2.33 Mat Thickness versus Time for Mat to Cool to 80 °C (Spreading Mix Temperatures 135, 105 °C).....	50
Figure 2.34 Effect of Compaction Temperature on Percent of Air Voids.....	52
Figure 3.1 Research Plan.....	58
Figure 3.2 Mechanical Model For Consolidation Process.....	64

Figure 3.3 Length of Drainage Path during Consolidation Process.....	66
Figure 3.4 Time versus Percentage of Consolidation (vl. Deformation).....	68
Figure 3.5 Mix Behaviour During Compaction Process.....	71
Figure 3.6 Elastic Materials (Linear and Non-linear).....	75
Figure 3.7 Anelastic Materials.....	76
Figure 3.8 Stress-Strain Curve.....	76
Figure3.9 Stress-Strain Curve for Different Types of Materials.....	77
Figure3.10 Time-Strain Relationship for Elastic, Plastic and Viscous Materials ...	78
Figure 3.11 Typical Creep Curve.....	80
Figure 4.1 Pavement Cross-section.....	88
Figure 4.2 Measuring the Density of Granular Layer (picture 1).....	94
Figure 4.3 Measuring the Density of Granular Layer (picture 2).....	94
Figure 4.4 Filling Paver with the Asphalt Mix.....	95
Figure 4.5 Laydown of the Asphaltic Layer (picture 1).....	95
Figure 4.6 Laydown of the Asphaltic Layer (picture 2).....	96
Figure 4.7 Laydown of the Asphaltic Layer (picture 3).....	96
Figure 4.8 First Stage in the Compaction Process of Asphalt Concrete Layer using Steel Roller (picture 1).....	97
Figure 4.9 First Stage in the Compaction Process of Asphalt Concrete Layer using Steel Roller (picture 2).....	97
Figure 4.10 Second Stage in the Compaction Process of Asphalt Concrete Layer using Pneumatic Roller (picture 1).....	98

Figure 4.11 Second Stage in the Compaction Process of Asphalt Concrete Layer using Pneumatic Roller (picture 2).....	98
Figure 4.12 Measuring of Asphalt Concrete Density during Compaction.....	99
Figure 4.13 Measuring Final Density of Asphalt Concrete (density after last pass).....	99
Figure 4.14 Schematic of Nuclear Gage.....	102
Figure 4.15 Corrected Density versus Number of Passes for First Layer of HL-8..	107
Figure 4.16 Corrected Density versus Number of Passes for Second Layer of HL-8.....	107
Figure 4.17 Corrected Density versus Number of Passes for HL-3.....	108
Figure 5.1 Consolidation Device.....	115
Figure 5.2 Final Set-up of Creep Device after Modification.....	117
Figure 5.3 Marshall Compactor.....	118
Figure 5.4 Marshall Molds and Specimens.....	119
Figure 5.5 Density versus Number of Blows for HL-8.....	121
Figure 5.6 Density versus Number of Blows for HL-3.....	121
Figure 5.7 Contact Area between Steel Compactor and Compacted Layer.....	123
Figure 5.8 Confining Medium Using Metal Mold.....	125
Figure 5.9 Confining Medium Using Asphalt Mix Ring.....	125
Figure 5.10 Confining Medium Using Sand.....	126
Figure 5.11 An Example of the output Results of the Creep Test	127
Figure 5.12 Asphalt Concrete Specimen before and after Compaction.....	129
Figure 5.13 Asphalt Specimen Coated with Insulating Sheet.....	132

Figure 6.1 Analytical Approach (Model I).....	142
Figure 6.2 Permanent Strain versus Time for Mix HL-8 (@ T = 115 °C, σ = 165 kPa, D = 2.241 gm/cm ³).....	146
Figure 6.3 Permanent Strain versus Time for Mix HL-3 (@ T = 115 °C, σ = 165 kPa, D = 2.227 gm/cm ³)	147
Figure 6.4 Corrected Field Density of First Layer of Mix HL-8.....	152
Figure 7.1 Recoverable and Permanent Deformation during Resilient Modulus Test.....	163
Figure 7.2 Resilient Modulus Results versus Water Content for Granular A.....	165
Figure 7.3 Compaction of Test Specimen	166
Figure 7.4 Resilient Modulus Test	167
Figure 7.5 Typical output of the Resilient Modulus Test.....	168
Figure 7.6 Strain of Confined Creep Test.....	170
Figure 7.7 Strain of Unconfined Creep Test.....	171
Figure 7.8 Relationship between the Strains of Confined and Unconfined Creep test.....	171
Figure 7.9 Stress-strain Relationship for Mix HL-8 at 25°C.....	173

LIST OF APPENDICES

APPENDICES	196
Appendix A. Permanent Strain Results For Mixes HL-8 and HL-3	197
Appendix B. Preliminary Analysis of Elastic and Plastic Strains Results for Mixes HL-8 and HL-3	206

CHAPTER ONE

INTRODUCTION

1.1 General

Asphalt Concrete Pavement is the most common type of pavement used around the world. Huang (1993) reported that *“As of 1990, there are about 2.2 million miles of paved roads in the United States, of which 94% are asphalt surfaced”*. In Canada, the total length of the paved roads in 1995 was 317,920 two lane equivalent kilometres, most of which are flexible pavements (TAC, 1995). Flexible pavement usually consists of bituminous surface layer/layers built over base course and subbase course materials, which are constructed on top of a compacted subgrade (Figure 1.1). Among these layers the top layer is constructed with high quality materials where the intensity of the stress applied by the wheel load is high. As the depth increases, the intensity of the stress decreases and materials with lower quality are usually used. This concept of design allows the construction of an economic pavement cross-section that can achieve the design requirements.

Unexpected deterioration of the asphalt concrete (AC) pavement is a serious problem facing highway engineers. The actual service life for most asphalt roads is usually less than the expected design life. Consequently, an additional budget is required to bring these roads back to an acceptable level of serviceability. Although these roads might be designed using different methods, constructed based on different experiences in different weather conditions, and used by different traffic levels, the types of pavement

Figure 1.1 Typical Flexible Pavement Cross-section

distress, which cause the deterioration of the pavement performance are almost the same in all cases (Abd El Halim, 1984).

1.2 Long-Term Performance of Asphalt Concrete Pavement

The long-term performance of asphalt concrete pavement can be affected by many factors. These include environmental conditions, traffic characteristics, characteristics of pavement mix and its ingredients, characteristics of the different layers of pavement cross-section, and the quality of construction and compaction processes. The pavement performance is usually evaluated several times during its service life to define its rate of deterioration. This rate is used to determine the overall quality of the addressed pavement cross-section (design process, material characteristics, and construction process) and to

determine the maintenance level required to operate this pavement at an acceptable level of serviceability. A good evaluation of the roads network condition enables the highway agencies to maximize benefits gained from the available budget (Garber and Hoel, 2002; Yoder and Witczak, 1975). In addition, this periodical evaluation allows for refining the specifications and procedures of road design and construction based on the actual performance monitored in the field.

There are four criteria that can be used for pavement evaluation process (Garber and Hoel, 2002):

- Pavement roughness.
- Pavement distress.
- Pavement deflection.
- Skid resistance.

Using these criteria, the concept of the Present Serviceability Index (PSI) was used to correlate the quality of the pavement to the road roughness, cracking, patching, and rutting (Yoder and Witczak, 1975).

The minimum value of PSI is zero, corresponding to a very poor pavement, and its maximum value is 5, corresponding to a pavement in very good condition. Practically, the maximum PSI value that can actually be achieved in the field is approximately 4.5, which is reached directly after the initial construction. On the other hand, the minimum PSI limit should not be allowed to drop below 2.5 for major roads and 2.0 for minor roads. Below these limits, rehabilitation or reconstruction works should be applied to bring these roads back to an acceptable level of serviceability.

The typical relationship between PSI and pavement age (or accumulated traffic) can be presented graphically as shown in Figure 1.2. The figure shows that the maximum value of the PSI (referred to as initial serviceability index, P_i) is achieved directly after the initial construction. This value then decreases gradually with time, while the rate of reduction of PSI, represented by the slope of the curve, increases with time. When the PSI reaches its minimum acceptable level (referred to as terminal serviceability index, P_t), major maintenance work (rehabilitation) should be undertaken to increase the PSI again to an acceptable level. After the rehabilitation, the PSI decreases again with time. If after a number of rehabilitation cycles the rate of deterioration of the PSI is so high that the benefit from rehabilitation process is small relative to its cost, a reconstruction should take place.

Figure 1.2 Pavement Life Cycle.

1.3 Importance of Asphalt Concrete Compaction

The compaction process has been recognized by many researchers as one of the most important factors that affects the long-term performance of asphalt concrete pavement. Marker (1972) stated that *“the compaction and densification of asphalt mixtures are the most important construction operations with regard to the ultimate performance of the completed pavement, regardless of the thickness of the course being placed”*. Noel (1977) reported that *“the single most important construction control that will provide for long term serviceability is compaction”*. Scherocman and Martenson (1982) stated that *“the air void content or density of the mix is probably the single most important characteristic of performance under traffic”*.

During the compaction process, the asphalt concrete mix changes from a non-cohesive to a cohesive state. Through this process the asphalt concrete layer gains its characteristics and strength which cannot be improved after completing the compaction. It should be noted that if a road was constructed with sufficient layer thicknesses and using high quality materials but at the same time had a poor compaction process, the pavement will dramatically fail to achieve its objectives. As a result of the high importance of the compaction process, a great deal of attention was given to it. Many studies were carried out to investigate the factors affecting this process and how this process can be improved, and the equipment used to carry out this process.

One of the major achievements during the last decade was the development of a new technology of compaction. Two rollers, belonging to this technology, Asphalt Multi-Integrated Roller (AMIR) and Hot-Iron Process Asphalt Compaction (HIPAC), were designed and manufactured (Abd El Halim et al. 1993; Rickards et al. 1998, 1999).

Several field tests indicated that both AMIR and HIPAC produce AC pavements with superior mechanical properties in fewer passes than those achieved by conventional compaction equipment. A detailed description of this new technology and the conventional technology is presented in the next chapter. This technology allowed the introduction of a new concept for explaining the compaction process, which considers the viscous characteristics of the asphalt mix during compaction and represents the compaction process by the consolidation theory used for fine-grained soil. A detailed description of this new concept is presented in Chapter 3.

It should be noted that the compaction process of asphalt concrete pavements is the broad subject of this research work. The behaviour of the asphalt mixes was defined in the lab under conditions similar to those occurring in the field during compaction. Then, using this behaviour the compaction process was modelled. The problem definition and objectives of this research work are presented in the next sections.

1.4 Problem Definition

Compaction of AC pavements is a very complex process, where its product is affected by a wide range of factors. These factors include characteristics of supporting layers (subgrade, subbase, and base), characteristics of the AC layer (compacted layer), temperature during compaction, environmental conditions, compaction load, load duration, compactor shape, and number of passes (Hughes, 1989; USACE, 2000). Considerable research has been carried out to study the effect of these factors on the quality of the compaction process. Although these studies have led to considerable improvements in the quality of the compacted asphalt concrete layer, the information

gained from most of them can be regarded merely as general guidelines. This information was based mostly on studies that were carried out under a specific set of conditions, which may not be encountered in many practical projects. It should be noted that the main reason for using this type of study is the absence of a test method or a framework that can be used to describe the behaviour of the asphalt mix itself during compaction.

As a result, the current procedures of pavement compaction are based on constructing one or more roller test strips (trial sections) with the same characteristics of the actual project (same mix, thickness, rolling equipment, supporting layer, etc.) in order to define the combination of compaction variables that can achieve a pavement with the required characteristics (USACE 2000). Although these test strips help select a good combination of the compaction variables, this combination might not be the optimum one. In addition, the construction process of many test strips requires additional budget, which increases the total cost of the project.

Thus, an accurate modelling of compaction that can predict the final density of the mix as a function of the main compaction variables (characteristics of the AC layer, temperature during compaction, compaction load, load duration, etc.) would greatly enhance the compaction process and lead to improving the long-term performance of the pavement. In addition, this model can save the cost of constructing several test sections. Using this model, the final density of the mix can be predicted prior to the actual compaction process in the field, and therefore, the optimum combination of the compaction variables can be selected for field compaction. The next sections summarize the objectives of this research and describe the organization of the thesis.

1.5 Research Objectives

Based on the above discussion, the main objectives of this research are summarized as follows:

- To investigate field compaction of asphalt concrete mixes and define its limitations.
- To review the consolidation theory and examine its applicability in modelling field compaction of asphalt concrete mixes.
- To explain the behaviour of asphalt concrete mix during compaction.
- To carry out an experimental investigation to investigate the behaviour of the asphalt mixes during compaction.
- To develop a model that can predict the compaction quality as a function of the main compaction variables.
- To verify the results through the design of a number field compaction tests.

The research plan required to achieve these objectives is illustrated in Figure 1.3.

A detailed description of this research plan is presented in Chapter 3.

1.6 Thesis Organization

This research work is organized as follows:

- Chapter 1 introduces the importance of the research work, research objectives, and thesis organization.
- Chapter 2 presents a review of the compaction process including its importance, mechanical characteristics of the asphalt concrete mix and their relationship to the compaction process, the available compaction technologies, and the factors affecting compaction process.

Figure 1.3 Research Plan

- Chapter 3 presents the research work required to achieve the objectives of this research. The analytical investigation required for studying and explaining the behaviour of asphalt concrete mixes during the compaction process is described.
- Chapter 4 provides a detailed description of monitoring process for one of the compaction projects.
- Chapter 5 presents the experimental investigation of the behaviour of asphalt concrete mix during compaction.
- Chapter 6 presents the analysis of the results of the experimental investigation in order to determine the behaviour of the asphalt mix during compaction. This behaviour was then correlated to the data monitored in the field to develop a compaction model.
- Chapter 7 defines the characteristics of the supporting layers for the monitored project. This information, in conjunction with the results from the experimental investigation (asphalt concrete characteristics), was used to develop an alternative compaction model.
- Finally Chapter 8 presents a summary, findings and recommendations.

CHAPTER TWO

ASPHALT CONCRETE COMPACTION

2.1 Introduction

Compaction is the process by which the air voids in asphalt concrete mix are reduced to their design values to improve the mechanical characteristics of the asphalt mix and gives it the ability to behave in the field as expected (Hughes 1989; Finn and Epps 1980). Many researchers have reported compaction as the most important factor affecting the long-term performance of flexible pavement (Marker, 1972; Noel, 1977; Scherocman and Martenson, 1982). As mentioned in Chapter one, a wide range of factors affects the output of the compaction process. These factors have to be optimized in order to produce a pavement meeting the expected performance along its service life.

This Chapter presents an extensive review of the compaction process of asphalt concrete mixes. Subjects included in this review are: objectives and importance of the compaction process, characteristics of the AC mix and their relation to the compaction process, historical development and characteristics of different types of the compaction technologies (conventional technology and new technology) and finally factors affecting the compaction process.

2.2 Objectives and Importance of Asphalt Concrete Compaction

The main objective of the compaction process is to give a final product that meets users requirements by (Finn and Epps, 1980; Pagani et al., 1999):

- obtaining design density with minimum variation,
- achieving a smooth surface for comfort riding,

- enhancing mix strength by achieving the required aggregate particle contact,
- providing an impermeable layer by providing crack-free asphalt surface, and
- providing a durable layer by reducing binder oxidation, which can be reduced by decreasing the percentage of air voids in the mix to their design value.

Significant amount of research was carried out in order to investigate the role of the compaction process in improving the quality of the flexible pavement. This research was mainly concentrated on studying the effect of the percentage of air voids on characteristics of the asphalt concrete mix, including stability, fatigue strength, shear and tensile strength, durability, and permeability. An overview of this research is presented below.

2.2.1 Stability

Stability is considered as an empirical measure for the strength of asphalt concrete mixes. It is defined as the resistance to deformation of an asphalt concrete mix when subjected to traffic loadings under a variety of environmental conditions (Finn and Epps 1980). Mix stability is measured in the laboratory by applying a constant rate of deformation, which is 50.8 mm per min (2 inch per min), diametrically on a cylindrical specimen of 101.6-mm (4-inch) diameter and 63.5-mm (2.5-inch) height until failure (Figure 2.1). The maximum load that occurs during this test is called stability, recorded in N or lb, and the total amount of deformation is called flow, recorded in units of 0.01-inch (Garber and Hole; 2002). Many studies were carried out to investigate the effect of the compaction process on the stability. Marshall (1948) indicated that as the density increases, the mix stability increases. Lefebure (1965) reported that as the density increases from 92% to

100% of the design value, mix stability at 60° C (140° F) increases from about (100 lb) 445 N to more than (1400 lb) 6225 N. Thereafter, and using data obtained from Lefebure (1965) study, McLeod (1967) investigated the effect of the compaction process on the corresponding pavement thickness requirements. In this investigation the modulus of stiffness for the asphalt mix at different levels of compaction was calculated using the following formula;

$$\text{Modulus of stiffness (lb/in}^2\text{)} = 40 * \text{Stability(lb)} / \text{Flow(0.01")} \dots\dots\dots(2.1)$$

Then, by using Burmister's procedure for two-layer elastic system and the results obtained from the above formula, the full depth pavement layer was calculated at different levels of compaction. The pavement cross-section during this analysis was assumed to be one asphaltic layer laying directly over a subgrade layer with a 9,000 lb wheel load applied on the surface of the asphaltic layer. California Bearing Ratio (CBR) for the underlaying layer was assumed to be 3%. CBR is used to indicate the strength of the tested material relative to the strength of a standard material, which is crushed stone (Youder and Witczak, 1975). Table 2.1 summarizes this analysis giving the results of the Marshall test (stability, and flow), calculated modulus of stiffness (using equation 2.1), and the required pavement thickness for each level of compaction using Burmister's procedure. The relationship between the relative density and the required pavement thickness is also illustrated in Figure 2.2. These results illustrate the effect of the compaction process in reducing the design pavement thickness, where increasing the

relative density from 93.7% to 96.4% reduces the required thickness by about 55% and this reduction in thickness reaches 68% when percent of density reaches 100%.

Figure 2.1 Marshall Test Machine
(Source: Durham Geo Enterprises web site)

Table 2.1 Influence of Degree of Compaction on Pavement Thickness Requirements

(Source: Mc Leod, 1967)

Percent of density (%)	Marshall stability, (lb) N	Flow, (0.01") mm	Modulus of stiffness, (psi) MPa	Pavement thickness, in (cm)
100*	(8,950) 39,196	(11) 0.28	(32,500) 220.6	(12) 4.7
99.1	(7,825) 34,269	(11.5) 0.29	(27,200) 184.6	(14) 5.5
96.4	(6,075) 26,605	(12) 0.30	(20,300) 137.8	(17) 6.7
95.1	(4,925) 21,568	(14) 0.36	(14,100) 95.71	(22) 8.7
93.7	(4,200) 18,393	(20) 0.51	(8,400) 57.02	(38) 15

* Density obtained by compacting a cylindrical specimen 60 blows on each face by Marshall Compactor

Figure 2.2 Effect of Relative Density on Required Design Thickness

2.2.2 Fatigue Strength

Fatigue is the phenomenon of repetitive load-induced cracking due to a repeated stress or strain level below the ultimate strength of the material (Yoder and Witczak 1975). Several studies have investigated the relationship between the percentage of the air voids in the AC mix and its fatigue life. The results of these studies concluded that:

- As the percentage of the air voids increases the fatigue life of the asphalt concrete mix decreases (Pell and Taylor, 1969).
- For each one percent increase in air voids, the fatigue life of the asphalt concrete mix decreases by 35% or more (Epps and Monismith, 1969).
- The fatigue life of asphalt concrete mix increases from five to sixfold when the compaction percentage changes from 90% to 100% (Bell et al., 1984).
- The construction-induced cracks resulting from steel compactor can decrease the fatigue life of asphalt concrete pavement by 50% or more (Abd El Halim and Hass, 1995)

2.2.3 Tensile and Shear Strengths

Asphalt concrete mix should have the ability to resist the development of cracks as well as the permanent deformation through the asphalt layer. Increasing tensile strength can prevent the crack initiation at the bottom surface of the pavement layer under the effect of the wheel load (Figure 2.3) and the crack due to thermal effect (Garber and Hoel, 2002; Youder and Witczak, 1975). Tons and Krokosky (1963) indicated that increasing the percentage of the air voids inside the asphalt concrete mix would decrease the tensile strength, and would reduce the effective cross-section that resists tensile force.

Rutting is a common type of asphalt concrete distress. It results from the accumulation of the permanent deformation that occurs in the wheel path under the effect of repetitive traffic loads (FHWA, 1995). One of the major causes of this type of distress can be attributed to inadequate shear strength of the AC layer. The shear strength is achieved by selecting a good aggregate and binder characteristics for the asphalt mix as well as conducting a good compaction process to ensure good interlock between the aggregate particles (FHWA, 1995; Rickards et al., 1999). Considerable research has been carried out at Carleton University in order to develop a shear-testing machine that can evaluate the shear properties of asphalt concrete pavement in the field. The first prototype of this machine was developed in 1995, called Carleton In-situ Shear Strength Test CISSST (Abdel Nabi et al., 1995). In 2000, an advanced prototype was developed, called In-Situ Shear Strength Test InSiSST (Goodman et al., 2000). Figures 2.4.a and 2.4.b illustrate the CISSST and InSiSST respectively. To test the pavement using this machine, a circular plate is glued to the pavement surface, then a torque is applied on this plate until failure occurs in the pavement (see Figure 2.5). Through this test the torque and the angle of rotation at failure are recorded and used to calculate the shear strength as well as shear modulus.

Figure 2.3. Tensile and Compressive Strength in Flexible Pavement

(a) Carleton In-situ Shear Strength Test (CISSST)

(b) In-Situ Shear Strength Test (InSiSST)

Figure 2.4 Shear Testing Machine

Figure 2.5 Testing Pavement Surface by CISSST/InSiSST

2.2.4 Durability

The durability of AC pavement is defined as its long-term resistance to the effects of ageing and to the abrasive action of traffic (Finn, 1967). The main factor affecting the durability of asphalt concrete pavement is the rate of hardening of the asphalt cement. Bright and Reynolds (1962) reported that *“Hardening involves a loss in penetration, a loss in ductility, an increase in softening point, and an increase in viscosity”*. These changes in the binder characteristics due to hardening make the binder brittle and reduce pavement service life (Bright and Reynolds, 1962). The pavement reaches the end of its service life when the penetration value of the binder reaches 20 to 30 (Hubbard et al., 1937). It should be noted that the penetration test is carried out by applying a 100-gm load on a standard needle which is left to penetrate a binder specimen for 5 Sec at 25° C (77° F) (Garber and Hoel, 2002). The unit of penetration is 0.1 mm, which implies that a penetration of 60 means that the needle penetrates the binder specimen 6 mm during the penetration test. Goode and Owings (1961) investigated the effect of the percentage of air

voids in the asphalt mix on the rate of hardening of the binder. They calculated the rate of hardening by determining the retained penetration of the binder at 25° C (77° F) after four years from construction as a percentage of the binder penetration at 25° C (77° F) immediately after rolling. Figure 2.6 illustrates the output results of this study.

Figure 2.6 Effect of Initial Pavement Air Voids on Change in Binder Penetration
(Source: Goode and Owings, 1961)

2.2.5 Permeability

Permeability can be defined as the ability of water and air to pass through the asphalt mix (Hughes, 1989). Permeability has been reported as a significant factor that can affect the durability of asphalt concrete layer and its susceptibility to moisture damage (Marker, 1967). As the permeability of the mix increases, water can infiltrate into the mix, therefore, reducing the bond between the binder and aggregate. The aggregate loss causes a major type of pavement distress known as the stripping. In addition, increasing the percentage of air inside the mix increases the rate of binder oxidation and subsequently reduces pavement durability, which may affect the long-term performance of the pavement layer (Zube, 1962). Several research efforts have been carried out in order to investigate the effect of compaction on asphalt concrete permeability. Zube (1962) reported that the permeability for a well-compacted AC mix with relative density 97% or more is 600% less than the poor compacted layer with relative density 92%. Figure 2.7 illustrates the relationship between the relative compaction and field permeability as reported by Zube (1962).

Although it is clear from the above discussion that, as the percentage of the air voids decreases, all the characteristics of the asphalt mix are improved. However, there is a minimum limit for this percentage below which the mix-bleeding problem may occur. This problem usually happens at high temperature weather, especially for mixes with a low percentage of air voids. The binder volume increases at high temperatures. If there was not enough room available to allow for binder expansion, some of the binder would exit to the pavement surface causing skid traffic problems (Yoder and Witczak, 1975). In order to avoid bleeding problems, most of the mix design methods have reported that the

percentage of air voids has to be between 3 and 5% (Finn and Epps 1980). This range can achieve the required mix properties mentioned above (Stability, Fatigue, Tensile and shear strengths, Durability, and Permeability) without causing bleeding to the asphalt mix.

Figure 2.7 Permeability – Relative Compaction relation for Ten Different Projects

(Source: Zube, 1962)

2.3. Compaction Technology for Asphalt Concrete Compaction

Currently, there are two types of technology used in pavement compaction, the conventional technology and new technology (Rickards et al.,1999). The following sections present the historical development of these technologies as well as the different stages required to carry out the compaction process using each technology.

2.3.1. Conventional Technology

Historical Development

In 19th century the common type of roller was a horse-drawn roller. In 1862 William Barford was granted a patent for increasing the weight of this type of roller by filling its cylinder with water (see Figure 2.8) (Parker and Hinman, 1960). Consequently, the first successful use of the steam traction force for developing a roller was in 1865. Regarding the first idea about steam rolling Parker and Hinman (1960) mentioned a story written by Hooley (1896). Hooley stated that *“Many claims have been laid as to the first idea of steam rolling, and the author has been many times informed by a man, now deceased, that he first gave the idea for a roller to a noted firm who had hitherto made only traction engines. He used to tell his story somewhat thus: ‘many years ago I was laying water pipes. We could not satisfy the surveyor as to reinstating his roads. One of these ugly great traction engines kept going past our work, so I told the driver one-day to just run his hind wheel up my trench. It was a very wet day and down went the trench. I filled it up and had the wheel on again, and so kept on. When the surveyor saw my trench he said he had never seen a better job and set about showing how the whole road could be so made,*

if a portion could be made with a comparatively narrow wheel; thus you see I started the idea of steam rolling."

After developing the steam traction engine in 1861, many trials were done in order to develop a steamroller. Gellerat & Co. of Paris, France (Figure 2.9) carried out one of those trials in 1865. Unfortunately, this roller did not succeed. The disadvantages of that one were: the pavement materials were pushed in front of the rollers wheels as a result of their small size, steering was small and ineffective, crown slope of the pavement could not be achieved due to the wideness of the rollers, and finally the rollers could not give a smooth finish to the road surface. Another two unsuccessful trials were carried out in 1865, by John Whittingham and Robert Douglas of Nantwick (Figure 2.10), and Morland & Co. (Figure 2.11).

Figure 2.8 Horse-Drawn Roller
(Source: Parker and Hinman, 1960)

Figure 2.9 Gellerat & Co. Roller
(Source: Parker and Hinman, 1960)

Figure 2.10 Morland & Co. Roller
(Source: Parker and Hinman, 1960)

Figure 2.11 Wittingham & Douglas Roller
(Source: Parker and Hinman, 1960)

The first successful steamroller was developed by Aveling & Porter (Figure 2.12) (Parker and Hinman, 1960). This type was three-wheel static roller and it was used to carry out the first compaction trial in London in 1866. After the success of the steamroller first development, several compactors were built and manufactured using the same idea (see Figure 2.13).

Figure 2.12 First Success Steamroller
(Source: Parker and Hinman, 1960)

**Figure 2.13 Different Shapes of Steamroller
Manufactured Between 1904 – 1934
(Source: Parker and Hinman, 1960)**

In 1904 the first motor driven roller (Figure 2.14.) was developed. Then, in 1927 a major change was made to its engine, which made this type of roller more economically than the steamrollers. The last steamroller was manufactured in United States in 1935 (Tunnickliff et al., 1974). Figure 2.15 illustrates the different types of motor rollers that were developed during the 1904 – 1934 period.

In 1878 the first steam tandem roller was produced in the United States by Andrew Lindelof (Figure 2.16). The major problem of this roller was its rocking motion, which produced unwanted waves in the rolled surface. Different trials were carried out in order to eliminate the effect of this rocking motion, but the major improvement actually happens after 1908 due to the use of a gasoline engine instead of the steam engine. The first version of this type, shown in Figure 2.17, was produced by Kelly-Springfield (Parker and Hinman, 1960).

Figure 2.14 The First Motor Roller
(Source: Parker and Hinman, 1960)

**Figure 2.15 Different Shapes of Motor Rollers
Manufactured Between 1904 – 1934
(Source: Parker and Hinman, 1960)**

**Figure 2.16 First Tandem Roller in U.S.
(Source: Parker and Hinman, 1960)**

**Figure 2.17 First Gasoline Tandem Roller in U.S.
(Source: Parker and Hinman, 1960)**

Both the three wheel and tandem rollers were the most common compaction equipment before World War II. The ballasted weight of the three-wheel roller was up to 13600 kg and 12700 kg for the tandem type (Geller, 1982).

Tractor drawn pneumatic tired rollers were used in the late of 1930s. This type of roller had nine tires arranged in a staggered way on two axle, the front axle had four tires and the rear one had five tires (Tunncliffe et al., 1974). The self-propelled pneumatic tire was introduced in early 1950s. By the middle of this decade this type of compactor was approved in many areas of the U.S. In the late 1950s, a variable air pressure device was inserted in this type of roller to enable the operator to change the tire pressure during the compaction according to the characteristics of the compacted layer. Figure 2.18 illustrates the shape of the self-propelled pneumatic roller.

In early 1960s, the vibratory roller was introduced and used to compact underlying granular soil. By the late 1960s, the vibratory rollers began to take popularity for compacting asphalt concrete, and the use of pneumatic roller declined (Tunncliffe et al., 1974; Geller, 1982). The first vibratory roller fabricated specifically to construct the

pavement layer was in about 1968. Figure 2.19 illustrates a roller having one vibrating drum and two pneumatic tired wheels (Tunncliffe et al., 1974).

Figure 2.18 Pneumatic Roller
(Source: Tunncliffe et al., 1974)

Figure 2.19 Vibratory Roller
(Source: Tunncliffe et al., 1974)

Current Stages and Equipment of Conventional Technology

The conventional technology for the asphalt concrete compaction uses three types; steel, vibratory, and pneumatic rollers. The conventional compaction process is generally accomplished in three stages, which are; the breakdown, intermediate, and finish rolling (Hughes, 1977; Geller, 1982). The first stage is carried out by applying the compaction load using a heavy steel/vibratory roller travelling over the top surface of each asphalt lift, to obtain the desired density. The second and third stages deal with smoothing out the surface of the asphalt layer using a pneumatic roller followed by a light steel roller.

The main characteristics for the conventional compaction equipment that are currently used are as follows:

Steel rollers

The weight of the static wheel rollers ranges from 3 to 14 tonnes (USACE, 2000). Bodrell (1957) measured the contact area between the drum of 10-12 ton three wheel rollers and the compacted layer to calculate the contact pressure corresponding to this area. The range of the contact pressure was from 276 to 414 kPa (40 to 60 psi). Geller (1982) commented on this range saying that, "*In my opinion the range is considerably wider than this in actual practice*". Rickards et al. (1999) reported that the compaction pressure resulting from a typical steel drum roller is about 1.38 MPa. In addition, the contact area between the steel drum roller and the pavement is about 7 to 15-cm long multiplied by the width of the steel drum, and the contact time between the steel drum and the asphalt layer, based on a 10 km/hr operating speed is about 0.036 sec per pass.

Pneumatic tire rollers

The compaction pressure that can be applied by this type of rollers ranges from 400 to 800 kPa (USACE, 2000). The value of this pressure and the contact area between the compactor and the pavement can be controlled by changing the inflation of the tire pressure.

Figures 2.20 and 2.21 illustrate recent photos, for steel and pneumatic rollers respectively. It should be noted that the main shape of both compactors is similar to their early version presented before.

The characteristics of the compaction equipment such as applied load, speed, drum diameter for the steel roller or tire pressure for pneumatic pressure as well as the number of passes depend mainly on the characteristics of the compacted layer and other compaction variables such as the supporting layer and environmental conditions.

Figure 2.20 Recent Steel Roller
(Source: Caterpillar web site)

Figure 2.21 Recent Pneumatic Roller
(Source: Caterpillar web site)

2.3.2 New Technology

Historical Development

Abd El Halim (1985; 1986) suggested that the mismatch of rigidities within the pavement system during the compaction process (steel roller, supporting layers, and asphalt concrete layer) as well as the geometry of the conventional roller contribute to the initiation of the construction cracks. Figure 2.22 illustrates the mismatch between the steel roller and the pavement layer. Although the paving industry has long recognised the existence of these cracks, it has also believed that they can be sealed-off by the pneumatic-tire rollers during the second stage of the compaction process. Geller (1982), stated “*pneumatic tire rollers do have the advantage of being able to eliminate hair cracks and shallow surface heat checks*”. However, closer investigations have shown that construction cracks are not sealed by the pneumatic-tire rollers or the traffic wheel tires. (Abd El Halim, 1984; 1985; 1986).

The first milestone in this new approach has been the development of a new compaction technology that can satisfy the following objectives (Abd El Halim, 1986):

- Reduce the modulus ratio between the compacted asphaltic layer and the roller ($E_{\text{roller}}/E_{\text{Pavement}}$).
- Increase the radius of curvature of the compactor to be infinite to match the radius of curvature at the bottom of the pavement layer (r_2/r_1).

These two items allow for a good matching of the rigidities and geometry between the compactor and the pavement layer during the compaction process, which can minimize the initiation of the construction cracks.

Figure 2.22 Steel Roller and Pavement System

The first prototype of the new compactor is called Asphalt Multi-Integrated Roller (AMIR) and consists of two steel rollers compounded in one unit by a rubber belt. By having a flat rubber material at the interface between the roller and the pavement, the modular ratio is very close to unity and the radius of curvature approaches infinity (Figure 2.23).

Figure 2.23 AMIR and Pavement System

Before building a full-scale prototype, small-scale models of AMIR and steel compactor were built at the Carleton University Laboratory (See Figures 2.24, 2.25). An experimental program was designed and carried out to compare the effect of the steel roller and AMIR on the compaction process. Results from this program proved that AMIR is more effective in performing the compaction satisfactorily while preventing the construction cracks (Abd El Halim, 1986).

Figure 2.24 Steel Roller "Small Scale Model"

**Figure 2.25 Asphalt-Multi Integrated Roller AMIR
"Small Scale Model"**

Having established the effectiveness of this new compaction technology in the lab, a full scale prototype was designed and fabricated as a joint venture between Carleton University and the National Research Council of Canada in 1989 (Figure 2.26). The purpose of this prototype was to investigate the applicability of AMIR in the field and its ability to prevent the construction cracks (Abd El Halim et al., 1993). Table 2.2 presents the main differences between conventional compactors and AMIR.

Figure 2.26 Asphalt Multi-Integrated Roller (AMIR)

Table 2.2 Main Differences between Steel Roller and AMIR

Criteria	Steel compactor*	AMIR*
- Radius	1.2 -1.5 m	Infinity
- Contact area** (between the roller and compacted layer)	$\cong 0.12-0.26 \text{ m}^2$	$\cong 3 \text{ m}^2$
- Contact time (based on 10km/hr speed)	$\cong 0.036 \text{ sec}$	$\cong 1.2 \text{ sec}$
- Stress applied to the asphalt layer	$\cong 1.38 \text{ MPa}$	$\cong 0.042 \text{ kPa}$

* as reported by Rickards et al., 1999.

** assuming drum width = 1.70 m.

Several field tests were carried out in Egypt and Canada to compare the conventional rollers and AMIR. In each field test, two side-by-side sections, were compacted using the AMIR on one section and the conventional rollers on the other. The following is a summary of results of these field tests (Abd El Halim et al., 1993).

- AMIR provided a crack free surface.
- The densities obtained by the AMIR, using fewer numbers of passes, were higher than those obtained by the conventional compactors.
- The AMIR compactor provided a higher indirect tensile strength than that obtained by the conventional rollers.
- The AMIR provided a fatigue life for the asphalt pavement greater than that provided by the conventional rollers.

More recently, a second generation of the AMIR technology was developed in 1998 by Pioneer Road Services, Australia, and was called the Hot-Iron Process Asphalt

Compaction (HIPAC). The major modification added in the HIPAC was using two units, with the same characteristics, for compaction instead of a single unit in the AMIR (Figure 2.27). This modification increased the advantages by reducing the number of passes required to achieve the desirable density, and increasing the roller's manoeuvrability.

Figure 2.27 Hot-Iron Process Asphalt Compaction (HIPAC)

Another set of field tests was carried out in Australia to investigate this new technology. The results of these tests supported the previous findings from the Egyptian and Canadian field tests. In particular, the permeability of the layers compacted by AMIR/HIPAC was less than that obtained by the conventional compactor (Rickards et al., 1999).

Current Stages and Equipment of New Technology

The three-compaction stages in the conventional technology were reduced to just one stage by this new technology. The HIPAC/AMIR is used to carry out the entire compaction process instead of the three different rollers used in the conventional technique. It should be noted that the AMIR was constructed for research purpose only to establish a new concept of compaction. The second generation, HIPAC, was built for industrial application after establishing and verifying this new concept. The characteristics of each unit of HIPAC are similar to the characteristics of AMIR (Table 2.2).

2.4 Factors Affecting Compaction Process

As mentioned before, compaction is a very complex process where its output is affected by a wide range of factors. Understanding and optimizing these factors would greatly enhance the asphalt mix characteristics and subsequently lead to the improvement of pavement's long-term performance.

Many research studies have investigated the factors affecting the compaction process. Among these studies are Dickson and Corlew (1970), Swanson et al. (1966), and

USACE (2000). Based on information from these studies, factors affecting the compaction process can be summarised as in Table 2.3. The following subsections aim at presenting an overview of these factors and how they affect the compaction process.

Table 2.3 Factors Affecting Compaction Process

Main Category	Sub-main Category
1. Mix and Material Characteristics	<ul style="list-style-type: none"> - Asphalt Cement - Aggregates - Mix Properties
2. Environmental Variables	<ul style="list-style-type: none"> - Air and Base Temperature - Mix Laydown Temperature - Wind Velocity and Solar Flux
3. Laydown Site Conditions	<ul style="list-style-type: none"> - Compacted Layer Characteristics - Supporting Layer Characteristics
4. Compaction Equipment	<ul style="list-style-type: none"> - Contact Pressure - Contact Area - Contact Time - Number of Passes - Shape

2.4.1 Material Properties

Asphalt cement

The asphalt cement content in the mix as well as its viscosity affect the compactability of the asphalt concrete mix (Hughes 1989, Finn and Epps 1980, and USACE, 2000). While a high percentage of the asphalt cement content in the mix may cause an excessive mat shoving under the roller, (see Figure 2.28), a low percentage increases the mix stiffness, which needs to increase the compactive effort (USACE, 2000; Hughes, 1989).

Figure 2.29 illustrates the influence of asphalt viscosity on the ease of compaction of paving mixtures (Mc Leod, 1967). It shows the density achieved using a high viscosity asphalt cement, at a given temperature, is lower than that achieved using a lower viscosity asphalt cement.

Figure 2.28 Mix Shoving

As indicated earlier in subsection 2.2.4, hardening involves an increase in viscosity. Bright and Reynolds (1962) illustrated that a high percentage of hardening takes place during the mixing process. They reported that as the mixing temperature increases the rate of hardening increases. Figure 2.30 presents the relation between the mixing temperature and the percent penetration retained as an indication for the hardening rate. It illustrated that up to 177 °C (350 °F), the loss in retained penetration is 2 percent for each 6 °C (10 °F) temperature and this rate becomes higher after 177 °C (350 °F). USACE (2000) indicated the importance of taking into consideration the degree of hardening, which occurs during the mixing process. As the degree of hardening increases, the stiffness of the mix increases and more compactive effort is needed. It should be noted that the rate of hardening depends on the characteristics of the binder. Consequently the mixing temperature has to be varying from one mix to the other depending upon the characteristics of the binder used in the mix.

Figure 2.29 Effect of Asphalt Viscosity on Compaction of Asphalt Concrete Mixes
(Source: McLeod, 1967)

Figure 2.30 Effect of Mixing Temperature on Percent Penetration Retained
(Source: Bright and Reynolds, 1962)

Aggregates

Efficiency of the compaction process involves the relative displacement of aggregate particles, which is a function of the internal forces developed within the mix composition. The amount of internal forces due to the aggregate particles is a function of different factors; the angularity, surface texture, gradation, and the ratio between filler and asphalt content (Hughes 1989). Angular particles and/or aggregates with a rough surface texture increase the resistance to the compaction effort (Santucci and Schmidt, 1962; USACE, 2000). The dense-graded material is the easiest material for compaction. The materials that have excess of fine aggregate between sieves No. 30 and No. 50 or between No. 40 and No. 80 are unstable under the roller during the compaction process (shoving problem). This is referred to the lack of internal cohesion (Hughes, 1989; USACE, 2000). On the other hand the importance of the mineral filler in the mix was discussed by Santucci and Schmidt (1962). Figure 2.31 illustrates the effect of the filler to asphalt ratio on the theoretical maximum percent density. The optimum filler to asphalt ratio was found to be 0.17. Finn and Epps (1980) reported that using a smaller percentage of the filler (3 to 6%) in the mix can achieve the required filler to asphalt ratio of 0.17 and subsequently enhances the compaction process.

Figure 2.31 Effect of Filler to Asphalt Ratio on Percent Theoretical Maximum Density
(Source: Santucci and Schmidt, 1962)

Mix Properties

The efficiency of the compaction process can be affected by properties of the mix such as; asphalt cement content in the mix, ratio between the filler and asphalt cement, and the fluids content in the mix (USACE, 2000; Hughes, 1989). The first two were previously discussed. USACE (2000) reported that increasing the percentage of the fluids content (moisture and asphalt cement) in the mix decreases the efficiency of compaction. This percentage has to be less than 0.5 percent from the total mix weight. Mixes that have excessive moisture content tend to be unstable during compaction and are difficult to compact.

2.4.2 Environmental Variables

The asphalt concrete mix consists of air, aggregate, and asphalt cement. The characteristics of the asphalt cement are highly dependent upon temperature. At high temperatures the viscosity of the asphalt cement decreases, which facilitates the compaction process. The laydown of the mix usually takes place at a temperature ranges from 120 -150 ° C (250 - 300 °F). The laydown temperature is the temperature of the mix after the paver screed passes over it. Through the time after laying down the mix, and based on the environmental conditions, the temperature of the mix decreases gradually during the compaction process, causing the asphalt cement viscosity, and subsequently, the mix stiffness to increase. At a given temperature any more application for the compactive effort may cause a mix failure depending upon the characteristics of the asphalt mix as well as its ingredients. This temperature is known as the minimum compaction temperature. Therefore, it is important to determine the rate of mix cooling during the compaction process and take the adequate precautions to carry out the entire compaction process at a suitable temperature and before the minimum compaction temperature is reached. Dickson and Corlew (1970) reported the environmental factors that affect the rate of mix cooling as follows; layer thickness, air temperature, base temperature, mix laydown temperature, wind velocity and solar flux. They developed a series of curves, which determines the time available for compaction (Figures 2.32 and 2.33). These curves were developed under the following conditions: 17.9 km/hr (11.1 miles/hr) wind velocity, 50Btu/ft²/hr solar flux, 80 °C (175 °F) minimum mix temperature suitable for compaction, and it is assumed that the base temperature is equal to the air temperature.

Figure 2.32 Mat Thickness Versus Time for Mat to Cool to 80 °C
 (Spreading Mix Temperatures 150, 120 °C)
 (Source: Dickson and Corlew, 1970)

Figure 2.33 Mat Thickness Versus Time for Mat to Cool to 80 °C
(Spreading Mix Temperatures 135, 105 °C)
(Source: Dickson and Corlew, 1970)

Air and Base/underlying layer Temperature

Heat from the mix is transferred to the air at the upper surface and the underlying layer at the lower surface. The rate of mix cooling, therefore, depends upon the temperature of both the air and under laying layer. As the temperature of these surfaces increases the rate of mix cooling decreases. Dickson and Corlew (1970) assumed that the temperature of the underlying layer is equal to the air temperature, which may not be the actual case in the field.

Mix Laydown Temperature

As mentioned before, the laydown temperature is defined as the temperature of the mix after the paver screed pass over it. As this temperature increases the time available for compaction increases. This can be illustrated by numerical example using Figure 2.32. For a layer thickness of 50-mm, air temperature of 15 °C, and laydown temperature of 120 °C, the available time for compaction is about 12 minutes. If the laydown temperature increases to 135 °C, the available time for compaction will increase to about 19 min. This factor can improve the compaction process for the thin layer compacted in cold weather where the time available for compaction can be increased. It should be taken into consideration that there is a limit for increasing the lay down temperature, which is controlled by hardening process that happens during the mixing process as mentioned earlier. Also, Parker and Hinman (1960) investigated the effect of increasing the mix temperature during the compaction process on the percentage of air voids. Figure 2.34 illustrates the results of this study.

Although it is clear that increasing the temperature gives more time for the compaction process, which can improve the quality of the compacted layer, however in some cases and according to the characteristics of the mix and its ingredients, it may cause a difficulty in the compaction process. This difficulty is due to the instability of the mix at very high temperatures, which makes the mix move under the compaction load in the horizontal direction rather than in the vertical direction. Therefore, the laydown temperature should not be fixed for all mixes but rather dependent upon the mix characteristics and the applied compaction load.

Figure 2.34 Effect of Compaction Temperature on Percent of Air Voids
(Source: Parker and Hinman, 1960)

Wind Velocity and Solar Flux

As the wind velocity increases the actual air temperature decreases. Therefore, wind velocity has to be taken into consideration, particularly during the compaction of thin layers that is carried out in cold weather.

Solar flux is defined as the amount of radiant energy available from the sun. On sunny days, the rate of mix cooling is less than that on cloudy days. Also, on sunny days, the temperature of the supporting layer is higher than that on cloudy days, which can reduce the rate of mix cooling.

2.4.3 Laydown Site Conditions

Laydown site conditions include characteristics of the compacted layer and the underlying layers.

Compacted layer characteristics

Several characteristics of the compacted layer have been recognized as factors that can affect the efficiency of the compaction process. These characteristics include layer thickness, uniformity of the compacted layer, and relationship between the maximum aggregate sizes used in the mix and the layer thickness (USACE, 2000).

Dickson and Corlew (1970) show that the rate of cooling increases as the thickness of the compacted layer decreases. USACE (2000) reported that for a less than 5-cm (2-inch) thin asphalt layer the rate of cooling is very high especially when the compaction is carried out in cold weather. On the other hand the rate of temperature loss decreases as the thickness of the compacted layer increases. The effect of the layer thickness on the rate of mix cooling can be seen in Figures 2.32 and 2.33.

USACE (2000) reported that it is difficult to achieve a uniform density when different pavement thicknesses are compacted at the same time. This may occur during overlaying an old pavement with rutting.

The last factor is the relationship between the maximum aggregate size and compacted layer thickness. USACE (2000) reported that as the maximum aggregate size increases the efficiency of the compaction process decreases. Therefore, it was suggested that the maximum aggregate size suitable for the compaction process is half of the pavement layer thickness.

Characteristics of supporting layer

Many studies have been carried out to investigate the effect of the supporting layer on the compaction process. Swanson et al., (1966) investigated the effect of the base support stiffness on density and stability of the compacted asphalt concrete. Both steel and rubber tire compactors were used to compact the asphalt concrete layer over three different types of base layer. Results from this study indicated that the effect of the base support stiffness on densities and stabilities of asphalt concrete layer is small. In general, more support stiffness gives slightly higher densities and stabilities. On the other hand, Kari (1967) and Marker (1967) mentioned the importance of using a firm base in order to achieve a higher density.

2.4.4 Compaction Equipment

Compaction equipment affects to a large extent the compaction process. In general, the variables of these equipment include; contact pressure, contact area, and contact time between roller and compacted layer. Selecting the proper combination of these variables during the compaction process would greatly enhance the characteristics of the compacted layer.

Contact pressure

Compaction process is carried out by applying a stress on the upper surface of the compacted layer. This stress is referred to as the contact pressure. The magnitude of the contact pressure has to be more than the mix strength otherwise, no density change would occur. On the other hand, the value of the contact pressure should not be too high,

otherwise the mix will move in the lateral direction rather than in the vertical direction and, therefore, no densification occurs. It should be noted that the mix strength is not fixed for all types of mixes but varies according to the mix ingredients. Therefore, a proper magnitude of the contact pressure has to be applied according to the mix characteristics in order to achieve a proper compaction process.

Contact Time.

Rickards et al. (1999) mentioned the importance of increasing the contact time between the roller and the compacted area. This importance arises because during the compaction process the mix temperature is relatively high, which makes the mix behave as a viscous material. The behaviour of this type of materials is time dependent, and consequently, by increasing the time of loading, the density increases.

USACE (2000) recommended reducing the roller speed to its minimum value in order to increase the contact time between the roller and the compacted layer.

2.5 Summary

This chapter involved presenting the importance and objectives of the compaction process, mechanical characteristics of the asphalt concrete mixes and its relation to compaction process, different types of compaction technology and the factors affecting this process. The review of the previous research work that has been carried out in the field of asphalt concrete pavement indicates several deficiencies. The previous research dealt with studying the effect of a specific factor(s) on the quality of the compaction without paying adequate attention to describe the behaviour of the mix itself during

compaction. In addition, there is no standard procedure available that can be used to determine the behaviour of the mix during compaction conditions. Furthermore, no compaction model is available in order to be used to predict the final density of the compacted layer as a function of the main compaction variables.

The next chapter will present the research work required to achieve the objectives of this research work as well as an analytical investigation to study and explain the behaviour of asphalt concrete mixes during compaction.

CHAPTER THREE

RESEARCH PLAN AND

ANALYTICAL INVESTIGATION

3.1 Introduction

The importance of investigating the behaviour of asphalt concrete pavement during compaction and the need to model the compaction process itself was identified through the previous Chapters. The main purpose of the current Chapter is to describe the research plan required to achieve the objectives of the research work presented in Chapter one. Also in this Chapter an analytical investigation was carried out to study the behaviour of asphalt concrete mix during compaction. The analytical investigation started with reviewing the consolidation theory for fine-grained soil to examine its applicability to model the compaction process. Then the different behaviours of the engineering materials were examined as well as the characteristics of the mix ingredients. Finally, and based on the findings of the analytical investigation, the behaviour of asphalt concrete mix during compaction was identified.

3.2 Research Plan

The objectives of the research work were presented previously in Chapter one. The research plan required to achieve these objectives is shown in Figure 3.1 and described below.

Figure 3.1 Research Plan

A. Review of compaction process of asphalt concrete mix (Chapter 2)

This step involved reviewing the previous research carried out in the area of asphalt concrete compaction. The topics covered in this part include the importance and objectives of the compaction process, mechanical characteristics of the asphalt concrete and its relation to the compaction process, the different types of compaction technology, and the factors affecting this process. It should be noted that this step was carried out in the previous chapter.

B. Analytical investigation (Chapter 3)

The ultimate goals of this step were to study and explain the behaviour of the asphalt concrete mix during compaction and select a suitable tool that can determine this behaviour. In order to achieve these objectives, the following tasks were included in this step:

- Reviewing the consolidation theory for fine-grained soil and examining its applicability in modelling the process of asphalt concrete compaction.
- Studying the different behaviours of engineering materials from stress-strain-time point of view, and the characteristics of the mix ingredients under compaction conditions.
- Based on the findings of the previous two tasks, the behaviour of asphalt concrete mix during compaction was assessed.

C. Field Monitoring Process (Chapter 4)

This step dealt with monitoring the field compaction process of asphalt concrete mixes. The data collected during this phase were used, in conjunction with the output results of

the experimental investigation presented in the next step, to model the compaction process as presented in Chapters 6 and 7. These data were:

- Pavement cross-section (number of layers and thickness of each layer).
- The characteristics of the asphalt concrete mixes and the characteristics of the supporting layers.
- The characteristics of the compaction equipment (types of rollers, weight, etc.).
- Compaction process parameters (number of passes, density after each pass, mix temperature during each pass, contact area between roller and the compacted layer, etc.)

D. Experimental investigation (Chapter 5)

Through this step and based on the results of the analytical investigation presented in step “B”, a procedure for testing the asphalt concrete mixes, under the same conditions of compaction process, was developed. This procedure was then used to define the characteristics of the asphalt concrete mixes monitored previously in the field. This step included the following tasks:

- Establishing a test procedure to define the characteristics of asphalt concrete mixes under compaction conditions.
- Designing an experimental program to determine the characteristics of those mixes monitored in the field.
- Implementation of the experimental program.

E. Modelling the compaction process based on experimental and field results,

Model I (Chapter 6)

In this stage, the results of the experimental investigation were used to define the behaviour of asphalt mix during compaction. Then this behaviour, in conjunction with the field monitoring data, was used to model the compaction process. The main tasks of this part were:

- Using the results of the experimental investigation to define the behaviour of the asphalt concrete mixes as a function of the main compaction variables (density, stress, time of loading and temperature).
- Using the field data monitored in step “C” to correlate mix behaviour in the lab with that during the field compaction.

It should be noted that the variables included in the compaction model were density, compaction stress, time of loading, and mix temperature.

F. Modelling the compaction process using existing software, Model II (Chapter 7)

In this step, an existing computer program was used to develop an alternative compaction model. The main difference between Model I and Model II is that the second model takes into consideration all variables included in the first model in addition to the characteristics of the supporting layers. The stages included in this step were:

- Defining the characteristics of the granular layers monitored in the field.
- Representing the characteristics of the different layers of pavement cross-section (asphalt layers and supporting layers) as well as the compaction load in the computer software.

- Running the software and examining the deformations developed in the pavement layers under the applied load. Then, from this output, the new density of the pavement layer can be calculated.
- Comparing the results obtained from this model against the monitored field data in order to evaluate its accuracy.

G. Summary, Conclusions and Recommendations (Chapter 8)

Finally, this step presents a summary, the main findings, and recommendations of this research.

3.3 Consolidation Theory for Fine-Grained Soil

The theory of consolidation was first introduced by Terzaghi (1925). Consolidation is defined as the reduction in the volume that may happen for a fully saturated soil of low permeability due to escaping pore water when an external load is applied on it for a long period of time (RRL, 1952).

3.3.1 Consolidation Process

The consolidation process can be illustrated by using the mechanical model presented in Figure 3.2. This model consists of a vessel filled with water and covered with perforated piston, which rests on a number of springs. In this model the deformation in the horizontal direction is restrained by the vessel, springs represent soil particles, and the perforated piston represents the permeability of the soil. If any external vertical load is applied on the piston the following stages will happen (RRL, 1952);

i. Initial stage (instantaneously after applying the load)

In this stage, the entire load is carried by water in a form of increased pore water pressure and the force acting on the springs is equal to zero. Consequently, no deformation will happen in the springs.

∴ At time = t_o

$$\Delta = 0,$$

$$U = P$$

Where

t_o = initial time ($t = 0$),

Δ = piston settlement (spring deformation),

U = pore water pressure, and

P = external vertical pressure.

ii. Intermediate stage

After a small period of time and under the effect of the vertical load the water will begin to escape through the perforated piston. Consequently, a part of the vertical load will transfer gradually to the springs and a settlement will happen in the springs.

∴ At time = t ($t_o < t < t_f$)

$$\Delta > 0,$$

$$0 < U < P$$

Where

t_f = final time ($t = \infty$)

a. First Stage

- Time = 0
- Pore water pressure = External stress
- Vertical deformation = 0

b. Intermediate Stage

- Time = t
- Pore water pressure < External stress
- Vertical deformation = Δ

c. Final Stage

- Time = ∞
- Pore water pressure = 0
- Vertical deformation = Δ_f

Figure 3.2 Mechanical Model For Consolidation Process

iii. Final stage

The intermediate stage continues until water stops exiting from the perforated piston due to the transference of the entire load to the spring. At the final stage the piston settlement reaches its maximum value, all the external forces are resisted only by the springs, and the pore water pressure is equal to zero.

∴ At time = t_f

$$\Delta = \Delta_f,$$

$$U = 0$$

Where

$$\Delta_f = \text{final settlement}$$

3.3.2 Consolidation Theory: one-dimensional method

Based on the previous discussion, the consolidation theory for the fine-grained soil can be described as follows (RRL, 1952):

- When an external load acts on a fully saturated soil of low permeability, the entire load is initially carried by the water in the voids (in this case the pore water pressure is equal to the external stress) and no volume change happens.
- After a small period of time and under the effect of the vertical load, the water will escape through the permeable layer/layers that border the fine layer (see Figure 3.3). Consequently, a settlement will take place in the low permeable layer and part of the vertical load will be transferred gradually to the particles of this layer
- Water escape will continue causing a gradual increase in the settlement until all the load transfers to the soil itself causing also the pore water pressure to vanish.

Figure 3.3 Length of Drainage Path During Consolidation Process

For developing a mathematical model to represent the consolidation process for fine-grained soil the following assumptions were applied (RRL, 1952):

- The soil is homogeneous and saturated.
- Permeability coefficient is constant throughout the consolidation process.
- Darcy's law is valid.
- Volume change happens due to water escaping from the voids under the effect of the external load.
- The deformation occurs in the vertical direction only.

Based on the previous assumptions the following formula was developed to predict the settlement, for a fully saturated soil of low permeability, subjected to an external load at any time t (RRL, 1952).

$$S = m_v * \Delta\sigma * H * \left[1 - \frac{8}{\pi^2} \sum_{N=0}^{N=\infty} \left(\frac{1}{(2N+1)^2} * \exp(-(2N+1)^2 * \frac{\pi^2}{4} * \frac{C_v t}{H^2}) \right) \right] \quad (3.1)$$

$$m_v = \frac{\Delta e}{\Delta\sigma} * \frac{1}{(1 + e_o)} \quad (3.2)$$

$$C_v = \frac{K}{\gamma_w * m_v} \quad (3.3)$$

Where

S = settlement at any time t ,

m_v = coefficient of volume change,

Δe = change in voids ratio caused by change in pressure equal to $\Delta\sigma$,

e_o = voids ratio before increasing the stress,

- H = consolidating layer thickness,
 C_v = consolidation coefficient,
 K = permeability coefficient,
 γ_w = weight of unit volume for the water, and
 exp = base of natural logarithm.

Based on the previous equation the relation between the percentage of consolidation (vertical settlement) and the time can be represented as in Figure 3.4.

Figure 3.4 Time versus Percentage of Consolidation (Vl. deformation)

3.4 Description of Asphalt Concrete Compaction Using Consolidation Theory

As mentioned in Chapter two the ingredients of the asphalt concrete mixes are binder and aggregates; and the viscosity of the binder affects to a large degree the quality of the compaction process. By taking the viscous characteristics of the binder into consideration Rickards et al, (1999) proposed modelling the compaction process using the consolidation theory for fine-grained soil.

During the compaction process and under the effect of the compaction load the aggregate particles are reoriented (moved and rotated) in order to get closer together, the binder flows between the aggregate particles, and the percentage of the air voids in the mix is reduced to their designed value (Figure 3.5). As a result of the viscous characteristics of the binder, its response to load is time and temperature dependent (the different behaviours of engineering materials are discussed in detailed in Section 3.5). So, if the compaction load was applied for a short period of time, even at a high temperature, the binder will not have enough time to flow between aggregate particles and the response of the mix will be elastic more than viscous. On the other hand if the compaction load was applied for a long period of time the binder will have a time to flow between the aggregate particles and as the loading period increases the binder flow increases and more percentage of compaction takes place. This is similar to the consolidation process where in the fine-grained soil as illustrated before; the entire applied load is resisted only by the pore water and no permanent settlement will take place unless the loading period is enough for allowing water escape. And by increasing the loading period, the amount of escaped water increases and the percentage of permanent settlement also increases.

Based on the above discussion the best way to compact the pavement is to carry out the entire compaction process in one pass by increasing the contact time between the roller and the compacted layer. But since this way is not suitable from the practical point of view, it is recommended to increase the contact time between roller and compacted layer to its maximum value by increasing the contact area between the roller and pavement as well as reducing roller speed to its minimum value.

This approach was verified through a number of field tests in Canada and Australia. These tests involved using the conventional rollers, which apply a large load for a small period of time, and the AMIR and HIPAC rollers, which apply a smaller load but for a longer period. The results of these tests showed that both AMIR and HIPAC rollers achieve the required density for the AC pavement in fewer passes than that achieved by the conventional rollers. (Rickards et al., 1999).

Figure 3.5 Mix Behaviour During Compaction Process
(Source: Rickards et al., 1999)

3.5 Differences Between the Consolidation Theory and Compaction Process

Although the behaviour of the asphalt mixes during the compaction process could be described using the concept of consolidation process for fine-grained soil, the equations that were developed to calculate the settlement in the fine-grained soil during the consolidation process can not be used to estimate the settlement (change in density) that occurs in the asphalt mix during compaction. This is due to the following differences;

- Fine-grained soil is temperature independent. Therefore, the consolidation equation does not take the effect of temperature into consideration. On the other hand asphalt mixes are temperature dependent which raises the importance of considering the effect of temperature during studying its behaviour.
- Coefficient of permeability is assumed to be constant through the entire consolidation process, which is not the case during the compaction process. Where the mix permeability decreases as the number of compaction passes increases.
- In developing the consolidation equation Darcy' law was used to determine the amount of water which escapes from the soil, then the amount of volume change in the soil was developed based on this value (RRL, 1952). This case is not applicable in the compaction process since the volume change in the mix (permanent deformation) is based on the binder flow as well as the reorientation and movement of the aggregate particles in the same time.

As a result of the above differences the consolidation equations for fine-grained soils are not applicable to study the compaction process for asphalt concrete mixes. However, it should be noted that the description presented by Rickards et al., with the aid

of the consolidation theory, could be considered as a good starting point to understand the interaction between binder, aggregate, and load.

In order to properly model the behaviour of the asphalt mixes during compaction, the different behaviours of engineering materials as well as the previous research works that investigated the viscous characteristics of the asphalt mixes were studied in the following sections.

3.6 Behaviour of Engineering Materials

The engineering materials can be classified based on the relationship between the applied stress and the gained strain “stress-strain relationship”. This relationship is governed by the type(s) of the strain(s) produced upon applying the stress on the considered material. There are three types of strain called elastic, plastic, and creep strain. These types can occur separately or combined; depending on the type of the material, applied load, time of loading and temperature during the loading period.

3.6.1 Elastic and Plastic Strains

Elastic strain (ϵ_e) is defined as the strain that appears/disappears instantaneously once the forces that cause it are applied/removed (Meyers and Chamlal, 1999). The value of the elastic strain is directly proportional to the applied forces only, regardless of the loading duration. The materials that produce this type of strain are called elastic materials. The elastic materials may be linear or non-linear depending on the shape of the stress-strain relationship. If the relationship between the applied stress and the gained strain is linear, the material is called “linear elastic”; otherwise the material is called “non-linear elastic”

(Figure 3.6). It should be noted that there are some types of materials that return to their original shape, upon removing the applied force, through an unloading curve different from the loading curve. Such materials are called “anelastic” (Figure 3.7).

Plastic strain (ϵ_p) is defined as the strain that appears instantaneously once the forces that cause it are applied and will not be recovered upon removing the applied forces. Similar to elastic strain, the value of the plastic strain depends on the applied forces only, regardless of the loading duration. The materials that gain this type of strain are called plastic materials.

Some materials behave elastically until a certain stress limit called yield stress σ_y which is the value of the stress that results in a small specified residual strain (usually 0.002) upon unloading. After the level of yield stress the plastic strain component will take place. Figure 3.8 shows a common type of the stress-strain curve that illustrates the elastic zone, plastic zone, and yield stress. In this figure, if the applied stress is less than σ_y all the gained strain will disappear instantaneously once the forces that cause it are removed (elastic strain) and the material will return to its original shape. On the other hand, if the applied stress was increased to more than σ_y the gained strain will consist of elastic and plastic strains. The elastic strain component will disappear instantaneously upon removing the applied force but the plastic strain will remain forever. Figure 3.9 illustrates the stress-strain curves for different types of materials.

Figure 3.6 Elastic Materials (linear and Non-linear)

Figure 3.7 Anelastic Materials

Figure 3.8 Stress-Strain Curve

Rigid

Linear elastic

Rigid, perfectly plastic

Rigid, plastic with linear strain hardening

Linear elastic, perfectly plastic

Linear elastic, plastic with linear strain hardening

Figure 3.9 Stress-Strain Curve for Different Types of Materials

3.6.2 Creep Strain

Creep Strain (ϵ_c) is a time-dependant strain observed during a constant stress test at an elevated temperature (Shames and Cozzarelli, 1992). In this case, the material is called a viscous material and gains its strain gradually with time. For a given material the value of this type of strain depends on the test temperature, the time of loading, and the applied load. Figure 3.10 illustrates a comparison between the elastic, plastic and viscous (creep) strain.

Figure 3.10 Time-Strain Relationship for Elastic, Plastic and Viscous Materials.

Creep strain consists of two parts. The first part is called recoverable strain ϵ_{rc} , which disappears over an interval of time after removing the load. The second part is called irrecoverable strain ϵ_{irc} , which does not disappear after removing the load and remains forever causing a permanent deformation in the material. The behaviour of viscous materials is usually described using the creep curve (Figure 3.11). This curve represents the relationship between the gained strain versus the loading time for a specific level of stress, which is applied on the material at a given temperature. As a general case, the total strain at any time t during the loading cycle can be given by the following equation:

$$\epsilon(t) = \epsilon_e + \epsilon_p + \epsilon_c(t) \dots \dots \dots (3.4)$$

Where

$\epsilon(t)$ = total strain at any time (t) ,

ϵ_e = elastic strain,

ϵ_p = plastic strain, and

$\epsilon_c(t)$ = creep strain at time (t) .

The creep strain, as illustrated in Figure 3.11, can be divided into three stages. In first stage, called primary creep, the rate of the gained strain decreases continuously. In second stage, called secondary creep, the rate of strain remains constant. In third and last stage, called Tertiary creep, the rate of strain increases until creep rupture occurs.

Figure 3.11 Typical Creep Curve.

3.7 Behaviour of Asphalt Concrete Mix

Until recently, most research work in the field of asphalt pavements has focused on the design of new structures to resist specific load repetitions over a predetermined period of time. Therefore, most present analytical methods assume that asphalt concrete mixes act approximately elastic in the field. Many methods and computer programs were developed to design and analyze the behaviour of flexible pavement based on this assumption. One of the popular computer programs, based on this assumption, is Bitumen Structures Analysis in Roads (BISAR). Through this program the stresses and strains components developed under the effect of traffic loads, can be calculated at any point in the pavement cross-sections. Then, according to the output of this program a suitable thickness for each pavement layer can be selected. Although there are some research works that investigated the viscous characteristics of asphalt mix, using the assumption of linear elastics is still

more popular. Such an assumption (linear elastic) can be used to represent the asphalt concrete layer during its service life. During this stage (service life), the pavement layer is subjected to high speed loads coupled with high strength layer and relatively low temperature which makes the structure act as an elastic.

On the other hand, during the compaction process, pavement characteristics are totally different than its characteristics during the service life. The compaction process is carried out at a significantly high temperature and very low density and strength especially at the beginning of this process. These conditions make the pavement layer exhibit viscous and plastic behaviour in addition to the elastic strain, which is totally different from its behaviour during the service life. As a result, the assumption of linear elastic material properties cannot yield accurate results during the compaction stage.

Although, there is significant research work carried out to investigate asphalt compaction, most of the work did not deal with how the mix behaves during compaction. Furthermore, the work concentrated on studying the effect of individual factors on the end product (for example, studying the effect of mix temperature during compaction on the final density of the mix). Consequently, the findings of these studies can be used only as guidelines and they can not fully describe the mix behaviour during the compaction process.

3.8 Viscous Behaviour for Asphalt Concrete Mixes

According to the author's knowledge, there are no previous studies that investigated the viscous behaviour of the asphalt concrete mixes during compaction. The available studies that investigated the viscous characteristics were carried out, only, on well-compacted AC

mixes (i.e. mixes that represent the pavement during its service life), but it should be noted that the number of these studies is relatively small.

One of those studies was carried out by Shell Oil Company to develop a method for determining the stiffness of the AC mix (Yoder and Witczak 1975). Variables included in this method were temperature, duration of the applied load, percent of the air voids in the mix and the characteristics of the bitumen used in the mix.

Perl et al. (1983) investigated the visco-elasto-plastic properties of a compacted asphalt concrete specimens at a constant temperature of 25°C. A set of mathematical models were developed to calculate the strain components for the asphalt mixes. The variables included in this study were stress, loading times and the number of loading cycles.

Lu et al. (2000) modelled the visco-elasto-plastic behaviour of compacted asphalt mixtures taking the effect of temperature, between 20 - 50°C, into consideration. This study reported that the deformation that occurs in the compacted asphalt mix has four components and recommended treating the plastic deformation as a part of the viscoplastic deformation.

3.9 Behaviour of Asphalt Concrete Mix during Compaction

Based on the previous discussion, the behaviour of the asphalt concrete mix during the compaction process can be described as follows;

- As a result of the conditions of the asphalt concrete compaction (high temperature and low strength) the behaviour of asphalt concrete mix exhibits viscous, plastic, as well as elastic strain.

- When the compaction load is applied on the mix, four deformation components are developed in the asphalt layer. These components are; elastic, plastic, viscoelastic (recoverable creep), and viscoplastic (irrecoverable creep) deformation. The magnitude of each component, for specific mix, depends on the compaction conditions (magnitude of load, time of loading, temperature, density of the mix before compaction...etc.).
- After the first loading cycle (first pass) the elastic deformation disappears instantaneously, the viscoelastic deformation disappears over the time, and the final permanent deformation is equal to the plastic and viscoplastic deformation.
- The behaviour of the mix during the following number of passes will be the same like its behaviour during the first pass with the exception of the value of each component of deformation is changing for each compaction pass. This refers to the changes that occur to the mix characteristics through the compaction process (reduction in temperature and increase in density).
- The compaction process is completed through a specific number of passes, which depend on the conditions of the compaction process such as mix characteristics, roller characteristics, supporting layer characteristics,...etc. Where after each compaction pass the value of the accumulated permanent deformation increases until the required density is reached.

As mentioned early in this Chapter, the best tool for describing the characteristics of the viscous materials is the creep curve. This curve is established by applying a constant stress on a test specimen and monitoring the deformation with time. From this curve the elastic, plastic, and creep strains can be calculated. To make this curve represent the actual

behaviour of the mix in the field, the actual field conditions have to be simulated in the lab.

Chapter 5 presents detailed description of the suggested test procedure involving the variables required to investigate the mix behaviour in the lab.

CHAPTER FOUR

FIELD COMPACTION AND DATA COLLECTION

4.1 Introduction

The ultimate objectives of this research as mentioned before are, to establish a procedure for investigating the behaviour of asphalt concrete mix during compaction and to model the compaction process of asphalt concrete pavement in the field. As indicated in Chapter one, two different methods will be used to model the compaction process. The first method will be carried out by investigating the behaviour of asphalt concrete mixes in the lab under the same range of mix temperature, mix density, and compaction stress occurring in the field. Then this behaviour will be correlated with actual field measurements in order to develop a compaction model (for more detail see Chapter Six). On the other hand, the second method of compaction modelling will be carried out by using the advancement of the available computer program that can take the effect of the viscous characteristics of asphalt mix into consideration. In this way the characteristics of the asphalt concrete mixes during compaction will be used as well as the characteristics of the supporting layers as input data for the computer software. This software, as will be presented in Chapter Seven, can be used to predict the deformation that occurs in the asphalt layer taking its viscous characteristics into consideration. Then to establish the validity of using this software to simulate the compaction process, its output results will be compared with actual compaction data collected from the field. The above discussion can be summarized through the following steps:

- Monitoring one of the compaction projects in the field. The monitored data have to include all the variables of the compaction process during each compaction pass as well as the density after each pass. (This step is described in this chapter)
- Taking representative samples from each pavement layer (supporting layers as well as asphaltic layers).
- Investigating the characteristics of the asphalt concrete mixes, those taken from the field during the monitoring process, in the lab. The mix characteristics will be investigated under the same conditions of the field during the compaction process (i.e. under the same range of temperature, stress, and density). The details of this investigation is presented in the next chapter (Experimental Investigation)
- Using the output results of the experimental investigation to model the behaviour of the asphalt concrete mixes, and then developing a compaction model by correlating the mix behaviour investigated in the lab to its actual behaviour monitored in the field. (This is presented in Chapter Six)
- Investigating the characteristics of the supporting layers in the lab, and then using them (in addition to the characteristics of the asphalt mix, compaction load, and loading time) to represent the characteristics of the pavement cross-section, during compaction, in the computer software.
- Running the software, and comparing its output results with the compaction data collected from the field to validate this procedure in modelling the compaction process.

The purpose of this chapter is to describe the field monitoring stage. The first part in this chapter describes the monitored project then the following parts present the data collected from this project.

4.2 Project Description

In order to select this project, an arrangement with City of Ottawa (formerly Regional Municipality of Ottawa-Carleton (RMOC)) was negotiated for monitoring one of the compaction projects in the city, and taking the samples required for the laboratory testing and experimental investigation. As a result of a new construction project was selected, representative samples from each layer of the pavement were taken during the construction process.

4.2.1 Pavement Cross-section

As mentioned before, the cross-section of the flexible pavement usually consists of the subgrade layers, granular layers (subbase and base), and finally the asphaltic concrete layers. The selection of the materials and thickness for each layer depends on many factors such as traffic loads, traffic density, environmental conditions, and subgrade strength.

The pavement cross-section constructed for this project consists of; Subgrade layer, 450-mm granular layer B type II, 150-mm granular layer A, two asphalt concrete layers type HL-8 each one has 50-mm thickness, and finally 40-mm asphalt concrete layer type HL-3. Figure 4.1 illustrates the typical pavement cross-section for this project. The granular B type II and A are two types of the granular materials that are allowed to be

used as a supporting layers in province of Ontario, the gradation specification limits of these two types as described in Ontario Province Standard Specification (OPSS) are illustrated in Table 4.1. On the other hand, the abbreviation “HL” indicates a Hot Laid asphalt concrete mix. There are different types of HL mixes, which are included in OPSS. Among these types are HL-8 which can be used as a binder course (binder course is the second asphaltic layer after asphaltic surface layer) and HL-3 which can be used as a surface course. Tables 4.2 and 4.3 illustrate the specification limits for the gradation and mix properties for HL-3 and HL-8 respectively.

Figure 4.1 Pavement Cross-section

Table 4.1 Aggregate Gradation Specification Limits for Granular Layers

Sieve Size (mm)	Specification Limits (Granular B Type II)	Specification Limits (Granular A)
150.0	100	N/A
26.5	50-100	100
19	N/A	85-100 (87-100*)
13.2	N/A	65-90 (75-95*)
9.5	N/A	50-73 (60-83*)
4.75	20-55	35-55 (40-60*)
1.18	10-40	15-40
0.30	5-22	5-22
0.075	0-10	2-8 (2-10**)

* Where the aggregate is obtained from an iron blast furnace slag source

** Where the aggregate is obtained from a quarry or slag source

Table 4.2. Mix Properties for HL-8 and HL-3

Design criteria	Specification limits	
	HL-8	HL-3
- Marshall stability (N @ 60° C)	8000 (min.)	8900 (min.)
- Flow value (@ 3.5% air voids)	8.0 (min.)	8.0 (min.)
- Air voids in mixture (%)	4 ± 1	4 ± 0.5
- Voids in mineral aggregates (%)	13.9 (min.)	15.1 (min.)
- Retained stability (%)	68 (min.)	68 (min.)

Table 4.3 Specification Limits of HL-8 and HL-3

Sieve Size (mm)	Specification Limits HL-8	Specification Limits HL-3
26.5	100	100
19.0	94-100	100
16.0	77-95	100
13.2	N/A	98-100
9.5	48-78	75-90
4.75	30-55	50-60
2.36	21-54	36-60
1.18	12-49	25-58
0.60	6-38	16-45
0.30	3-22	7-23
0.15	1-9	3-10
0.075	0-6	0-5

4.2.2. Construction Process

The construction process of the pavement cross-section started with preparing and compacting the subgrade and followed by constructing the granular layers (granular A, and granular B type II). Table 4.4 presents the characteristics of the subgrade layer and Tables 4.5 and 4.6 present the gradation of the granular layers used in the construction.

After finishing the construction of the supporting layer, the paver laid down the first layer of the asphalt mix which is HL-8. Then the compaction process was carried out, to achieve the desired density of this layer, through three stages. The first stage was

done using two drum steel rollers, the second stage using the pneumatic rollers, and the last one using a lightweight steel roller. After finishing the compaction process and making sure that the percent of compaction meets the specification limits, the next two layers were laid down and compacted using the same way that was used for the first layer. Tables 4.7 and 4.8 present the characteristics of the asphalt concrete layers (HL-3 and HL-8) used in this project. During the construction process, samples from each layer of the pavement cross section were taken from the construction site in order to define their characteristics in the lab. Figures 4.2 through 4.13 illustrate the various stages of the construction process and data collection.

Table 4.4 Subgrade Characteristics

Criterion	Value
- Soil Type	Clay
- Liquid limit	47 %
- Plasticity index	25 %
- Soil Classification (according to AASHTO system)	A-7-6

Table 4.5 Aggregate Gradation for Granular Layers

Sieve Size (mm)	% passing (Granular B type II)	% passing (Gradation A)
37.5	100	100
26.5	89.6	100
19	N/A	92.3
13.2	N/A	79.8
9.5	N/A	66.3
4.75	41.6	48.9
1.18	18.6	22.4
0.30	11.0	11.4
0.075	7.3	6.1
Density (gm/cm ³)	2.2	2.19

Table 4.6 Mix Properties for HL-8 and HL-3

Design criteria	Mix properties (HL-8)	Mix properties (HL-3)
- Marshall stability (N @ 60° C)	14000	13350
- Flow value (@ 3.5% air voids)	11.9	11.1
- Air voids in mixture (%)	3.5	4.2
- Voids in mineral aggregates (%)	14.4	15.1
- Bulk relative density (g/cm ³)	2.450	2.430
- Maximum relative density (g/cm ³)	2.540	2.537
- Retained stability (%)	74	71

Table 4.7 Aggregates Gradation and Binder Contents for Mixes HL-8 and HL-3

Sieve Size (mm)	Gradation of HL-8 (% passing)	Gradation of HL-3 (% passing)
26.5	100	100
19.0	98.0	100
16.0	86.4	100
13.2	N/A	98.2
9.5	61.7	83.9
4.75	53.9	56.9
2.36	47.5	48.8
1.18	40.9	41.9
0.60	32.4	33.2
0.30	17.9	18.3
0.15	6.7	6.9
0.075	3.8	3.9
% AC	5.0 (PG 58-34)*	5.0 (PG 58 – 34)*

* PG (performance grade: This is a classification system was developed recently by Strategic Highway Research Project SHRP. In this system the binder is classified according to the temperature that can work effectively init. PG 58-34 means hat the binder can perform effectively between temperature ranges -34°C to 58°C .

Figure 4.2 Measuring the density of granular layer (picture 1)

Figure 4.3 Measuring the density of granular layer (picture 2)

Figure 4.4 Filling paver with the asphalt mix

Figure 4.5 Laydown of the asphaltic layer (picture 1)

Figure 4.6 Laydown of the asphaltic layer (picture 2)

Figure 4.7 Laydown of the asphaltic layer (picture 3)

Figure 4.8 First Stage in the compaction process of asphalt concrete layer using steel roller (picture 1)

Figure 4.9 First Stage in the compaction process of asphalt concrete layer using steel roller (picture 2)

Figure 4.10 Second stage in the compaction process of asphalt concrete layer using pneumatic roller (picture 1)

Figure 4.11 Second stage in the compaction process of asphalt concrete layer using pneumatic roller (picture 2)

Figure 4.12 Measuring of asphalt concrete density during compaction

Figure 4.13 Measuring final density of asphalt concrete (density after last pass)

4.2.3. Compactors Characteristics

As mentioned earlier the compaction process was carried out through three stages. The first, second, and third stages were carried out using a two-drum steel roller, pneumatic roller, and light two-drum steel roller, respectively. Tables 4.8 and 4.9 illustrate the characteristics of the two rollers used in the first and second compaction stages respectively.

Table 4.8 Characteristics of Steel Roller used in the First Compaction Stage

Criteria	Values
- Drum width	1700 mm
- Drum diameter	1300 mm
- Load of first drum	82.61 kN
- Load of second drum	86.39 kN
- Roller speed	5 km/hr

Table 4.9 Characteristics of Pneumatic Roller used in the Second Compaction Stage

Criteria	Values
- Number of wheel per axle	4
- Contact Pressure	490 kPa
- Contact area	600 cm ²
- Roller speed	5 km/hr
- Contact time	0.155 sec

4.2.4. Data and Measurements

The data measured during the monitoring process were as follows:

- Number of compaction passes
- Density after each pass
- Temperature after each pass
- Contact area between the compactor and the pavement surface during each pass.
- Degree of compaction for the supporting granular layers.

Field density after each pass

The accurate way for defining the density of the asphalt concrete layers is carried out by taking cores specimens from the field and determining their densities in the lab by measuring their weights and volumes. Practically this procedure is not suitable for measuring the density of the asphaltic concrete layer after each compaction pass. The reason of this is that during the compaction process, as mentioned before, the temperature of the asphalt mix is very high and its strength is low. As a result, if the coring process is carried out at such conditions all of the core specimens will fail. Therefore, the suitable way for monitoring the density of the asphalt concrete layer during compaction was found to be “the nuclear gauge method”. This method is included in the standards of American Society for Testing and Materials (ASTM) under the designation number of D 2950 – 91 (ASTM, 1994).

The nuclear gauge method is a non-destructive test that can be used for measuring the in-place density of bituminous mixtures by the attenuation of gamma radiation. In this

method, the source and detector can remain on the surface, or one of the source or detector is placed at a certain depth (up to 300mm) and the other one remains on the surface. Then the density is estimated by transmitting gamma rays into the mix and measuring at the detector, the amount of radiation reflected in a specific period of time. It should be noted that the density obtained by the nuclear gauge is relative and it is not the same as the density measured in the lab using a core specimen. So, the densities measured by the nuclear gauge method have to be modified with a correction factor. This factor is established by taking a set of readings by the nuclear gauge (after finishing the compaction process) at given spots, then cutting a set of core specimens at those spots and measuring their densities in the lab, and finally correlating the nuclear gauge readings to the laboratory densities to establish the required factor. Figure 4.14 illustrates a schematic of nuclear gauge (case of source and detector on the surface).

Figure 4.14 Schematic of nuclear gauge
(Source: USACE, 2000)

During the monitoring process the field densities were taken, after each pass, using the nuclear gauge standard method (see Figures 4.12 and 4.13). Table 4.10 illustrates these measurements for the different layers of pavement (HL-8 and HL-3). It should be noted that, as a result of the densities did not change during the third stage of compaction, the data of this stage are not included in Table 4.10. After finishing the compaction process a set of readings was taken using the nuclear gauge. Then, after the pavement temperature decreases to the level that allows for coring process, a set of core specimens were taken on the same spots which were tested using the nuclear gauge. The densities of the core specimens were determined in the laboratory using the test procedure included in ASTM (1994) under designation number of D 2726 – 93a. It should be noted that this process was carried out twice, once to establish the correction factor for mix HL-8 and the other one to establish the correction factor for mix HL-3. Table 4.11 presents the readings of the nuclear gauge as well as the corresponding core densities at the same spots.

Then, the correlation factors between the nuclear gauge readings and the core densities measurements were established by dividing their averages as follows;

$$\text{Correlation Factor for HL-3} = 2.368/2.372 = 0.998$$

$$\text{Correlation Factor for HL-8} = 2.412/2.394 = 1.008$$

Using the above two correction factors, all the nuclear gauge readings presented previously in Table 4.10, were corrected and summarized in Table 4.12. Also the

corrected densities versus the number of passes for first and second layer of HL-3 and the layers of HL-8 are illustrated in Figures 4.15, 4.16, and 4.17, respectively.

Table 4.10 Measured Field Density versus Number of Passes

Pass number	Measured field density (gm/cm ³)		
	layer # 1 (HL-8)	Layer # 2 (HL-8)	layer # 3 (HL-3)
Paver	2.044	1.997	2.067
1 (Steel roller)	2.284	2.259	2.290
2 (Steel roller)	2.327	2.325	2.329
3 (Steel roller)	2.370	2.339	2.337
4 (Steel roller)	2.354	--	--
1 (Pneumatic roller)	2.350	2.340	2.346
2 (Pneumatic roller)	2.343	2.346	2.352
3 (Pneumatic roller)	2.361	2.347	2.382
4 (Pneumatic roller)	2.352	2.361	2.344
5 (Pneumatic roller)	2.358	2.338	2.368
6 (Pneumatic roller)	2.403	2.358	--

Table 4.11 Gauge Reading versus Core Density for HL-3 and HL-8

Sample #	HL-3		HL-8	
	Gauge reading	Core density	Gauge reading	Core density
1	2.346	2.344	2.408	2.410
2	2.381	2.39	2.387	2.404
3	2.425	2.406	2.366	2.391
4	2.309	2.299	2.371	2.415
5	2.380	2.367	2.405	2.428
6	2.392	2.387	2.407	2.433
7	2.370	2.383	2.391	2.398
8	2.397	2.402	2.415	2.420
9	2.367	2.361		
10	2.357	2.342		
Averages	2.372	2.368	2.394	2.412
Standard Deviation	0.031	0.033	0.018	0.015

Table 4.12 Corrected Field Density versus Number of Passes

Pass number	Corrected field density (gm/cm ³)		
	layer # 1 (HL-8)	Layer # 2 (HL-8)	layer # 3 (HL-3)
Paver	2.060	2.013	2.063
1 (Steel roller)	2.302	2.277	2.285
2 (Steel roller)	2.346	2.344	2.324
3 (Steel roller)	2.389	2.358	2.332
4 (Steel roller)	2.373	--	--
1 (Pneumatic roller)	2.369	2.359	2.341
2 (Pneumatic roller)	2.362	2.365	2.347
3 (Pneumatic roller)	2.380	2.366	2.377
4 (Pneumatic roller)	2.371	2.380	2.339
5 (Pneumatic roller)	2.377	2.357	2.363
6 (Pneumatic roller)	2.422	2.377	--

**Figure 4.15 Corrected Field Density versus Number of Passes
for First Layer of HL-8**

**Figure 4.16 Corrected Field Density versus Number of Passes
for Second Layer of HL-8**

Figure 4.17 Corrected Field Density versus Number of Passes for HL-3

Temperature and Contact area

As mentioned previously, the temperature, the contact area and time are important variables that have to be included in the measured data. Table 4.13 presents the temperature, which was measured using a hand-held infrared thermometer, of all monitored asphalt layers at each compaction pass. On the other hand, Tables 4.14, 4.15, and 4.16 present the contact time and curve length between the compactor and the pavement layer for each compaction pass for first and second layers of HL-8, and layer of HL-3 respectively. It should be noted that, the contact time and area between the pneumatic roller and the compacted layer was given in Table 4.9. On the other hand, the contact time between the steel roller and the compacted layer was measured by marking a point on each drum of the roller and then counting, using a stopwatch, the contact time

between these points and the compacted layer. Finally, the contact curve length was calculated using the roller speed (Table 4.9) and the contact time.

Table 4.13 Temperature during Compaction Process

Pass number	Temperature °C		
	Layer # 1 (HL-8)	Layer # 2 (HL-8)	layer # 3 (HL-3)
Paver	148	146	150
1 (Steel roller)	139	136	138
2 (Steel roller)	131	128	128
3 (Steel roller)	123	124	120
4 (Steel roller)	116	--	--
1 (Pneumatic roller)	107	113	111
2 (Pneumatic roller)	102	106	103
3 (Pneumatic roller)	100	101	97
4 (Pneumatic roller)	98	97	92
5 (Pneumatic roller)	94	92	89
6 (Pneumatic roller)	89	93	85

Table 4.14 Contact Time, Curve, and Pressure during Compaction of HL-8 (1st layer)

Pass number	Contact Time (Sec)	Contact Curve Length (mm)	Pressure (kPa)
1 (1 st drum of steel roller)	0.203	282.4	172.08
1 (2 nd drum of steel roller)	0.163	226.9	223.96
2 (1 st drum of steel roller)	0.133	185.2	262.39
2 (2 nd drum of steel roller)	0.123	171.3	296.66
3 (1 st drum of steel roller)	0.110	152.8	318.02
3 (2 nd drum of steel roller)	0.103	143.5	354.13
4 (1 st drum of steel roller)	0.103	143.5	338.63
4 (2 nd drum of steel roller)	0.100	138.9	365.86

Table 4.15 Contact time, Curve, and Pressure during Compaction of HL-8 (2nd layer)

Pass number	Contact Time (Sec)	Contact Curve Length (mm)	Pressure (kPa)
1 (1 st drum of steel roller)	0.207	287.0	169.32
1 (2 nd drum of steel roller)	0.157	217.6	233.54
2 (1 st drum of steel roller)	0.133	185.2	262.39
2 (2 nd drum of steel roller)	0.120	166.7	304.84
3 (1 st drum of steel roller)	0.107	148.1	328.12
3 (2 nd drum of steel roller)	0.102	141.7	358.63

Table 4.16 Contact Time, Curve, and Pressure during Compaction of HL-3

Pass number	Contact Time (Sec)	Contact Curve Length (mm)	Pressure (kPa)
1 (1 st drum of steel roller)	0.200	277.8	174.92
1 (2 nd drum of steel roller)	0.160	222.2	228.70
2 (1 st drum of steel roller)	0.130	180.6	269.07
2 (2 nd drum of steel roller)	0.117	162.0	313.69
3 (1 st drum of steel roller)	0.110	152.8	318.02
3 (2 nd drum of steel roller)	0.105	145.8	348.54

Compaction degree of the underlying layer

The degree of compaction of the underlying layer as well as their water content affect their rigidities which affect the compaction process. Since the characteristics of the underlying layers are going to be used in Chapter 7 to represent the pavement cross section in the computer software, it is important to monitor these two variables (degree of compaction and water content).

The densities of the granular layers were obtained using the nuclear gauge method before laying down the upper layers. On the other hand, the water content was determined by taking field samples and drying them in an oven. The differences between the weight of the specimen before and after drying as a percentage of the dry weight represent the water content expressed as a percentage. The degree of compaction was found to be 99.2

and 99.8% for granular A and granular B type II, respectively. The water contents were 2.4 % and 2.3% for granular A and granular B type II, respectively (see Table 4.17).

4.3 Summary

This Chapter involved describing the field monitoring stage of the asphalt concrete compaction. The data collected in this stage include:

- Characteristics of the supporting layers.
- Characteristics of the asphalt concrete layers.
- Characteristics of the compactors.
- Temperature, time of loading, and compaction pressure during each compaction pass.
- Density after each compaction pass.

The data collected in this Chapter in conjunction with the results obtained from the laboratory investigation, in the next Chapter, will be used to model the compaction process of the monitored project.

Table 4.17 Water Contents and Density of Granular Layers

Criterion	Granular A	Granular B type II
Water content	2.4	2.3
Degree of compaction (%)	99.2	99.8
Density (gm/cm ³)	2.172	2.196

CHAPTER FIVE

EXPERIMENTAL INVESTIGATION

5.1 Introduction

This Chapter deals with the laboratory investigation of the behaviour of the asphalt concrete mixes under the same conditions of the compaction process occurring in the field. It should be noted that, there is no standard procedure available to test the asphalt mixes under such conditions (i.e. high temperature and low strength). Consequently, a test method has to be established initially.

This chapter is organized as follows:

- Development of a test procedure for investigating the behaviour of the asphalt mixes under the compaction conditions.
- Design and implementation of an experimental program to identify the behaviour of the two asphalt concrete mixes monitored in the field.

As mentioned before the output results of this investigation will be used in conjunction with the field data presented in the previous Chapter to model the compaction process.

5.2 Development of a Test Procedure to Investigate the Behaviour of Asphalt Concrete Mixes during Compaction

As mentioned before, there is no standard test procedure available to investigate the behaviour of the asphalt concrete mixes under conditions similar to those occurring in the

field during compaction. Consequently, in this part, and based on the findings of Chapter 3, a test procedure was established for this purpose.

The following findings were obtained from Chapter three:

- The compaction process is carried out at a high temperature and low mix strengths.
- At such conditions the mix exhibits plastic, viscous, as well as elastic characteristics.
- The creep curve is used to study material that exhibits a viscous characteristic.

As a result the test procedure has to be able to predict the creep curve of the asphalt concrete mixes under the same conditions of the compaction process. The creep curve is predicted by applying a constant load for a given period of time and recording the change in the vertical deformation that occurs in the materials through the time. After that, and in order to measure the recoverable deformation components, the applied load is removed and the reduction in the vertical deformation is recorded versus the time.

5.2.1 Test Equipment

The output information required from the test investigation is a relationship between the vertical deformation versus time under constant stresses. Such information can be collected using the consolidation device (see Figure 5.1). As shown, the consolidation device consists of loading plate, loading head, loading lever, and a dial gauge to measure the vertical deformation. The load transferred to the test specimen through the loading lever is ten times the load acting on the loading plate. For example if the load is 10 kg, the actual load that will stress the specimens will be 100 kg. This device is available in the Civil Engineering Laboratory at Carleton University.

a. Elevation

b. Side view

Figure 5.1 Consolidation Device

In this device, the vertical deformation versus time is recorded manually, where the vertical deformation is read from the dial gauge (See Figure 5.1) at specific time intervals. Although this manual recording is suitable for the original purpose of using this device, which is testing soil specimens whose deformation occurs over a relatively long period of time, the device is not suitable in the case of testing asphalt mixes under compaction conditions. The reason for this is that, under such conditions, the change in the vertical deformation of asphalt mix occurs rapidly and over a relatively short period of time, which cannot be recorded manually. To overcome this problem the original consolidation device was modified by replacing the manual system for data recording with an automatic one. A data acquisition system was installed to collect the vertical deformation data and transfer them directly to a computer system for storing the data over time. This modification allows the reduction of the time interval between the recorded data up to 0.05 second. Figure 5.2 illustrates the final set-up of the device after this modification.

Figure 5.2 Final Set-up of Creep Device after Modification

5.2.2 Specimens Preparation

Marshall compactor (Figure 5.3) is one of the most common compactors used to fabricate asphalt concrete specimens in the lab. The method of specimens fabrication using this compactor is included in the American Society for Testing and Materials (ASTM) standard test under designation number D1559 – 89. The fabricated specimens, using this method, are cylindrical in shape with standard dimensions of 4-inch (101.60-mm) in diameter and 2.5 ± 0.05 -inch (63.5 ± 1.27 mm) in height (Figure 5.4).

Figure 5.3 Marshall Compactor

In this method, the asphalt concrete mix is heated in an oven until it reaches the compaction temperature, which depends on the characteristics of the binder used in the mix. This temperature is the one required to produce a binder with kinematic viscosity equals to 280 ± 30 centistokes. For the two mixes under study (HL-3 & HL-8), the compaction temperatures are $147 \pm 3^{\circ}\text{C}$. After the mix reaches the compaction temperature, it is placed in a heated Marshall mold (see Figure 5.4 for Marshall molds). Then the molds filled with the mix are moved to Marshall compactor and compacted

using a 10-lb (4536-gm) hammer with a free fall of 18-in (457.2-mm) for a given number of blows for each specimen face. The number of blows is defined according to the project specification.

It should be noted that the number of blows that is stated in the project specification is the number required to prepare fully compacted asphalt specimens and it ranges from 50 to 75 blows per face. But in this research work the fabricated specimens have to be prepared at low densities to simulate the mix conditions during the compaction process, which required reducing the number of blows. So, in order to determine the number of blows required to achieve a specific density, the master curves were constructed for the mixes under study to define the relationship between the number of blows and the mix density.

Figure 5.4 Marshall Molds and Specimens

5.2.3. Density versus Number of Blows

In order to construct the relationship between the density and the number of blows, thirty specimens were prepared from each mix under study (HL-8 and HL-3) at ten different levels of compaction (three specimens per level). These levels were 0, 1, 2, 4, 6, 10, 20, 30, 45, and 60 blows per face. Table 5.1 and 5.2 illustrate the average density at each level of compaction for HL-8 and HL-3 respectively. In addition Figures 5.5 and 5.6 illustrate the density versus number of blows for HL-8 and HL-3 respectively.

Table 5.1 Density versus Number of Blows for HL-8

Number of blows	Density (gm/cm ³)	Number of blows	Density (gm/cm ³)
0	1.980	10	2.356
1	2.138	20	2.380
2	2.241	30	2.404
4	2.280	45	2.434
6	2.321	60	2.444

Table 5.2 Density versus Number of Blows for HL-3

Number of blows	Density (gm/cm ³)	Number of blows	Density (gm/cm ³)
0	1.978	10	2.341
1	2.131	20	2.363
2	2.227	30	2.376
4	2.280	45	2.393
6	2.320	60	2.418

Figure 5.5 Density versus Number of Blows for HL-8

Figure 5.6 Density versus Number of Blows for HL-3

5.2.4 Test Variables

The compaction process of asphalt concrete mixes is achieved over a given number of passes, which depends on the mix characteristics and the existing compaction conditions. In each pass and under the effect of the compaction load an increase in density (permanent deformation) is developed in the mix. But this increase in density is not constant for each of the compaction passes. Where the maximum gain in density is achieved during the first pass, this gain decreases gradually through the next passes until reaching its minimum value at the last pass. During the first pass the mix temperature has its maximum value and the mix density has its minimum value, which makes the internal resistance force of the mix very small. Then the temperature decreases and density increases gradually over the following number of passes, which increases the internal resistance of the mix against the compaction load. So, the increase in density resulting from each pass is less than the preceding one. Also, it should be noted that as a result of the inequality of the permanent deformation developed over each pass, the contact area and pressure between the compactor and the compacted layer is not constant through all the passes (see Figure 5.7).

Consequently, to address the behaviour of the asphalt mix over the entire compaction process the mix has to be tested under different temperatures, densities, and loads. The ranges of those variables should be equal to the actual range occurring in the field during the compaction process. In this research work and based on the monitored data presented in the previous chapter Table 5.3 summarizes the test variables as well as their ranges.

*Contact area = contact curve * drum width*

Figure 5.7 Contact Area between Steel Compactor and Compacted Layer

Table 5.3 Test Variables

Variable	Range
Temperature	80 – 150 °C
Density	2.01 – 2.38 gm/cm ³
Pressure	169 – 490 kPa

5.2.5 Pilot Testing

The purpose of the pilot testing is to examine a set of dummy specimens, before running the experimental program, in order to achieve the following objectives:

- Selecting the appropriate medium required to confine the tested specimen during the creep test.
- Verifying the description of compaction process presented previously in Chapter 3

The first part of this section deals with selecting the confining medium that can be used to confine the asphalt mix specimen during the creep test to best simulate the vertical deformation that occurs in the field during mix compaction. Three different confining media were tested to select the most appropriate one among them. These media are as follows:

- Metal mold with inner diameter equal to the outer diameter of the test specimens (see Figure 5.8).
- Rings fabricated from asphalt mix with inner diameter equal to the outer diameter of the test specimens. These rings were made by fabricating asphalt mix specimens with a 10-inch (254-mm) diameter then cutting a smaller cylindrical specimen with smaller diameter from those large specimens (see Figure 5.9).
- Metal mold, with a 10-inch (254-mm) internal diameter, filled with sand (see Figure 5.10).

Figure 5.8 Confining Medium Using Metal Mold

Figure 5.9 Confining Medium Using Asphalt Mix Ring

Figure 5.10 Confining Medium Using Sand

In this pilot test a total number of six specimens, of the mix type HL-3, were prepared at 2.280 gm/cm^3 density using 4 blows on each specimen side (Table 5.2). The specimens were divided into three different groups each one has two specimens. Then all the specimens were tested using the consolidation device at 128°C temperature and under 290 kPa pressure, which are equal to the compaction conditions during the second pass (Tables 4.13 and 4.16). The only difference between the testing conditions for all the groups was the confining medium, where the metal mold, asphalt mix rings and metal mold with the sand were used as a confining medium for first, second, and third group respectively.

The results of these tests indicated that the irrecoverable strain developed in the tested specimens has its minimum value when the metal mold is used as a confining medium. On the other hand, the irrecoverable strain has its maximum value when the

sand is used as a confining medium. Table 5.4 illustrates the value of the irrecoverable strain developed in each specimens at a loading time 0.25 sec, which is similar to the average loading time monitored in the field during the second pass. Figure 5.11 illustrates an example of the output results of the creep test (the case of sand confining medium).

Figure 5.11 An Example of the Output Results of the Creep Test

Table 5.4 Irrecoverable Strain for Different Confining Media

Confining Media	Specimen #	Initial density (gm/cm ³)	Irrecoverable strain	Average Irrecoverable strain
Metal	1	2.280	0.005	0.0055
	2		0.006	
Asphalt mix ring	3		0.006	0.007
	4		0.008	
Sand	5		0.015	0.016
	6		0.017	

In order to compare the performances of the mix tested under those three different confining media, the results of the irrecoverable strain were used to calculate the new density. The equation required for this calculation was derived as follows:

- Assume that a core specimen was taken from the field before compaction with the following characteristics (see Figure 5.12);
 - Mass = M
 - Height = h_o (this height equal to the thickness of the layer before compaction)
 - Core diameter = r
 - Volume = $V_o = h_o * \pi * r^2$
- After compaction, another core specimen was taken from the compacted layer. Since the compaction affects only the layer thickness (i.e. the surface area and the mix weight will remain constant), the height of this specimen “h” will be equal to the new thickness

of the compacted layer. So the characteristics of the core specimen which was taken after compaction are as follows;

- Mass = M
 - Height = h
 - Core diameter = r
 - Volume = $V = h * \pi * r^2$
- From the specimen characteristics before compaction, the initial density " D_o " can be calculated as follows;

$$D_o (gm / cm^3) = \frac{M(gm)}{V_o(cm^3)} \dots\dots\dots (5.1)$$

- On the other hand, the density of the specimen after compaction " D " can be calculated as follow;

$$D (gm / cm^3) = \frac{M(gm)}{V (cm^3)} \dots\dots\dots (5.2)$$

Figure 5.12 Asphalt Concrete Specimen before and after Compaction

- From equation 5.2

$$M = D * V$$

By substituting in equation 5.1

$$D_o = D * \frac{V}{V_o}$$

$$\therefore D_o = D * \frac{\pi * r^2}{\pi * r^2} = D * \frac{h}{h_o} = D * \frac{h_o - \Delta h}{h_o}$$

$$\therefore D_o = D * \left(1 - \frac{\Delta h}{h_o}\right) = D * (1 - \varepsilon)$$

where

Δh = reduction in specimen height due to compaction

ε = irrecoverable strain

$$\therefore D = \frac{D_o}{1 - \varepsilon} \dots \dots \dots (5.3)$$

It is important to realize that several assumptions have been made to simulate the field conditions during compaction in the lab. It is recognized that the asphalt layer during compaction is a multi-phase, multi-layer structure. As a result, the material in front of the roller and before compaction is in a semi solid, semi liquid state. Consequently, this layer tends to move excessively under the effect of the horizontal forces generated by the steel drums. However, the asphalt materials immediately behind the same drum have the property of a solid compacted material with very little lateral movement. Therefore, while

the author recognizes this characteristics it is assumed that this horizontal displacement is negligible with the confined conditions in the lab.

Using Equation 5.3, the strain data illustrated in Table 5.4 were used to calculate the new densities, which are presented in Table 5.5. On the other hand, the actual new density as monitored in the field after the second pass was 2.324 gm/cm^3 (see Table 4.12), which indicates that the sand is the most appropriate confining medium among the three media taken into consideration. Consequently, the sand will be used as a confining medium over the experimental program of this research work.

On the other hand the output results presented in Figure 5.11 confirm the description of the compaction presented in Chapter 3. As indicated in the figure, an instantaneous strain occurs in the tested specimens after its loading, and this strain increases through the time. Then after unloading the specimen an instantaneous reduction in the vertical deformation occurs. But, it should be noted from this figure that there is almost no further reduction in the vertical strain occurs through the time after the unloading.

Table 5.5 Density after Creep Test for Different Confining Media

Confining media	Density after creep test (gm/cm^3)
Metal	2.293
Asphalt mix ring	2.296
Sand	2.317

5.2.6 Achieving and Maintaining Test Temperature

Also it should be noted that the consolidation device is designed to test the materials at room temperature. But the asphalt mixes will be tested at a high temperature to simulate the conditions of the compaction process. So, in order to achieve and maintain the required temperature over the creep test the following procedure will be followed:

- Prior to the creep test, the tested asphalt mix specimens will be cured in an oven until its temperature reaches the required value.
- Then directly after removing each specimen from the oven, it will be coated with an insulating sheet to maintain its temperature at the required value over the entire test period. Figure 5.13 illustrates an insulated specimen.

Figure 5.13 Asphalt Specimen Coated with Insulating Sheet

5.3 Experimental Program

The main purpose of this experimental program is to define the behaviour of the asphalt concrete mixes monitored in the field. As indicated before, the conditions of the mix (temperature, and density) as well as the compaction load are not constant all over the compaction process. Consequently the effects of these changeable factors on the mix behaviour has to be taken into consideration. So, the behaviour of each mix will be defined by developing two equations. The first one deals with describing the elastic strain of the mix as a function of density, temperature, and stress. The second equation deals with describing the permanent strain developed in the mix as a function of time of loading, density, temperature, and stress.

5.3.1 Elastic Strain

To predict the elastic behaviour of the two mixes under study, a total of twenty-one cylindrical specimens from each mix, were prepared at three different levels of compaction. These specimens were divided into seven groups per mix; each one has three specimens. Each group was tested using the creep device according to the test conditions illustrated in Table 5.6. Before testing, the height of each specimen was measured.

Table 5.6 Test Conditions of the Experimental Program “case of elastic strain”

Group number	Specimen numbers	Level of compaction (# of blows)	Temperature (°C)	Stress (kPa)
1	1,2,3	2	115	165
2	4,5,6	2	80	330
3	7,8,9	2	115	330
4	10,11,12	2	115	495
5	13,14,15	30	115	330
6	16,17,18	0	115	330
7	19,20,21	2	150	330

The testing procedure for all the specimens under testing was as follows;

1. Curing the test specimens with the confining medium in an oven at the required test temperature until their temperatures reach the required test temperature.
2. Exiting the specimen from the oven and coating it with an insulating sheet (as illustrated in Figure 5.13) to maintain its temperature.
3. Mounting the specimen in the creep device, covering the upper face of the specimen with a 100-mm plate, adjust the loading head of the creep device to be in contact with the covering plate, applying the stress (165, 330, or 495 kPa) according to the test conditions listed in Table 5.6, and finally removing the applied stress.
4. Removing the tested specimen from the creep device and leaving it until reaching the room temperature and then measuring its density.

5. From the data recorded in the computer files, the reductions in vertical deformation after removing the applied stress will be calculated.

The results of this part of testing are illustrated in Tables 5.7 and 5.8 for HL-8 and HL-3 respectively.

Table 5.7 Results of Elastic Strain for Mix HL-8

Group number	Specimen numbers	Temperature (°C)	Stress (kPa)	Density (gm/cm ³)	Elastic strain (%)
1	1	115	165	2.258	0.45
1	2	115	165	2.271	0.43
1	3	115	165	2.262	0.47
2	4	80	330	2.261	0.47
2	5	80	330	2.269	0.50
2	6	80	330	2.271	0.45
3	7	115	330	2.287	0.82
3	8	115	330	2.301	0.78
3	9	115	330	2.291	0.77
4	10	115	495	2.315	0.98
4	11	115	495	2.340	1.02
4	12	115	495	2.371	1.07

Table 5.7 (continued) Results of Elastic Strain for Mix HL-8

5	13	115	330	2.410	0.68
5	14	115	330	2.422	0.72
5	15	115	330	2.417	0.72
6	16	115	330	2.181	0.82
6	17	115	330	2.199	0.87
6	18	115	330	2.202	0.80
7	19	150	330	2.336	1.33
7	20	150	330	2.340	1.32
7	21	150	330	2.351	1.32

Table 5.8 Results of Elastic Strain for Mix HL-3

Group number	Specimen numbers	Temperature (°C)	Stress (kPa)	Density (gm/cm ³)	Elastic strain (%)
1	1	115	165	2.282	0.40
1	2	115	165	2.263	0.38
1	3	115	165	2.269	0.42
2	4	80	330	2.281	0.40
2	5	80	330	2.272	0.40
2	6	80	330	2.279	0.45

Table 5.8 (continued) Results of Elastic Strain for Mix HL-3

3	7	115	330	2.318	0.72
3	8	115	330	2.301	0.72
3	9	115	330	2.299	0.73
4	10	115	495	2.360	0.97
4	11	115	495	2.357	1.00
4	12	115	495	2.341	1.05
5	13	115	330	2.451	0.63
5	14	115	330	2.441	0.67
5	15	115	330	2.439	0.63
6	16	115	330	2.210	0.75
6	17	115	330	2.201	0.80
6	18	115	330	2.205	0.75
7	19	150	330	2.342	1.28
7	20	150	330	2.349	1.23
7	21	150	330	2.361	1.20

5.3.2 Permanent Strain

To predict the permanent strain of the two mixes under study, a total number of twenty-one specimens, from each mix, were prepared using Marshall compactor. The specimens were divided into seven groups and tested according to the conditions listed in Table 5.9.

Table 5.9 Test Conditions of the Experimental Program “case of permanent strain”

Group number	Specimen numbers	Level of compaction (# of blows)	Temperature (°C)	Stress (kPa)
1	22, 23, 24	0	115	330
2	25, 26, 27	2	115	165
3	28, 29, 30	2	150	330
4	31, 32, 33	2	115	495
5	34, 35, 36	30	115	330
6	37, 38, 39	2	115	330
7	40, 41, 42	2	80	330

The testing procedure was carried out in a way similar to that used in predicting the elastic strain except for the following;

- The applied stress remained on the specimens until the rate of change of vertical deformation was negligible.
- The monitoring test data for analysis in this part are the vertical deformation occurring in the tested specimens versus time.
- It should be indicated that the monitored data during the creep test include the elastic as well as the permanent deformations. To separate this components for each specimen the following procedure was followed;
 - Developing a model that can predict the elastic strain of the mixes as a function of stress, temperature, and density. The data summarized in Tables 5.7 and 5.8 will be used to develop this model in the next Chapter.

- Using the model developed in the previous step to determine the amount of elastic strain for the specimens from 22 to 42 based on their testing conditions.
- Removing the elastic strain calculated in the previous step from the results of the creep tests.

The results of the permanent strain for the specimens 22 to 42 were presented in Appendix A.

5.4 Summary

Through this Chapter a test procedure to investigate the behaviour of asphalt concrete mixes under compaction conditions was developed. Then an experimental program was designed and implemented to investigate the behaviour of the asphalt concrete mixes monitored in the field. The output results of this investigation will be analyzed in the next Chapter to model the behaviour of these mixes during compaction.

CHAPTER SIX

MODELLING THE COMPACTION PROCESS BASED ON EXPERIMENTAL AND FIELD RESULTS (Model I)

6.1 Introduction

The main objective of this Chapter is to use the output data of the experimental program to determine the behaviour of asphalt concrete mixes during compaction. It has been mentioned before that the behaviour of the mix will be described by the components of the elastic and permanent strains that occur in the mix. Subsequently a compaction model was developed by correlating the results of the experimental program with the actual compaction data obtained from the field. These objectives were achieved by completing the following tasks:

- Using the Statistical Package for Social Sciences (SPSS) and the data of elastic strain shown in Tables 5.7 and 5.8 to develop statistically-based models that includes all test variables to predict the elastic strain (recoverable strain).
- Using the models developed in the previous step to calculate the value of elastic strain of specimens 22 to 42 of HL-8 and HL-3.
- Subtracting the value of the elastic strain calculated in the previous step from the data of the creep tests of specimens 22 to 42 to develop a set of curves for the two mixes under study (permanent strain = total strain – elastic strain). These curves describe the change of the permanent strain (irrecoverable strain) over time under different combinations of test variables.

- Using SPSS and the data of the permanent strain obtained in the previous step in order to develop a model that can predict the permanent strain of the mix as a function of compaction load, compaction time, density, and temperature.
- Modifying the permanent strain models developed in the previous step to represent the density instead of permanent strain, and then using these modified models (compaction models) to calculate the densities of mixes HL-8 and HL-3, under the same conditions monitored in the field for each compaction pass.
- Calibrating the output results of the compaction models by correlating the densities measured in the field with those calculated using the models.

Figure 6.1 illustrates the previous steps. Through the following sections a description of each step mentioned above will be presented for the two mixes under study.

* σ = stress, t = time, T = temperature, D = density

** see Table 5.9 for the different combinations of the test conditions

Figure 6.1 Analytical Approach (Model I)

6.2. Regression Analysis

Regression analysis method is one of the statistical techniques that can be used to find the best equation that can describe the trend of a specific variable (dependent variable) as a function of other set of variables (independent variables). The predicted equation is combined with the coefficient of determination R^2 that indicates the relationship between the predicted trend and the actual set of data. This factor ranges from 0 to 1. Younger (1972) stated that;

- If the value of R (correlation coefficient) is 0 (no relationship).
- If the value of R is +/- 0.25 (moderately weak relationship).
- If the value of R is +/- 0.50 (moderate relationship).
- If the value of R is +/- 0.75 (moderately strong relationship).
- If the value of R is +/- 1 (strong relationship).

6.3. Modelling Elastic Behaviour of Asphalt Mix

As mentioned before, the effect of three different variables were investigated on the elastic strain of asphalt concrete mix. These variables are stress, temperature, and density. Tables 5.7 and 5.8, in the previous chapter, summarize the elastic strain results obtained from the experimental tests carried out on mixes HL-8 and HL-3 respectively. As shown in these tables, the specimens of each mix were categorized in seven different groups. Each group was tested at a particular combination of test conditions.

Before incorporating all the results of each mix in one model that can predict its elastic strain as a function of the temperature, density, and stress. The general formula of the model was established initially through a preliminary analysis where the effect of

each factor on the elastic strain was investigated separately. This analysis was presented in detail in Appendix B and indicated that the general formula that can be used to determine the elastic (recoverable) strain as a function of density, temperature, and stress is as follows:

$$\epsilon_{recoverable} = C_1 \sigma e^{C_2 T} e^{D_o C_3} \dots\dots\dots(6.1)$$

Where

$\epsilon_{recoverable}$	= Recoverable strain
$C_1, C_2, \text{ and } C_3$	= Constants
σ	= stress (kPa)
T	= Temperature (°C)
D_o	= Density (gm/cm ³)
e	= Base of natural logarithm

Finally, SPSS was used to define the constants of Equation 6.1 for mixes HL-8 and HL-3 for each mix as well as the coefficient of determination R^2 . The output results of this software indicated that C_1 , C_2 , and C_3 for HL-8 were 3.68×10^{-5} , 0.0158, and -0.98 respectively and R^2 was 0.94. On the other hand C_1 , C_2 , and C_3 for mix HL-3 were found 2.45×10^{-5} , 0.0165, and -0.871 respectively and R^2 was 0.97. So the two equations that can predict the recoverable strain for the two mixes under study can be written as follows:

- For the Mix HL-8,

$$\varepsilon_{recoverable} = 3.68 * 10^{-5} \sigma e^{0.0158T} e^{-0.98D_o} \dots\dots\dots(6.2)$$

$$R^2 = 0.94$$

- For the Mix HL-3,

$$\varepsilon_{recoverable} = 2.45 * 10^{-5} \sigma e^{0.0165T} e^{-0.871D_o} \dots\dots\dots(6.3)$$

$$R^2 = 0.97$$

6.4. Permanent Deformation of Asphalt Mix

As indicated before, the permanent deformation developed in the asphalt concrete mixes will be determined by subtracting the elastic strain from the output results of the creep tests carried out on specimens 22 to 42. The experimental program was designed in order to study the effect of the temperature, density, and stress on the permanent strain for HL-3 and HL-8. As indicated in Table 5.9 the test specimens were divided into seven groups for each mix, each group examines the behaviour of the mix under particular combination of test conditions. It should be noted that each group includes three specimens tested under the same conditions.

Firstly, the elastic strain was calculated using Equations 6.2 and 6.3, for mix HL-8 and HL-3 respectively, under the seven conditions listed in Table 5.9. Then these strains were deducted from the raw data of the creep tests of specimens 22 to 42. This means that the data of each test after this deduction represent only the permanent strain versus time. Then the regression analysis technique was applied on the permanent strain data available for each group to establish the trend of their changing versus time. Figures 6.2 and 6.3

give an example for the permanent strain data versus time for mix HL-8 and HL-3 respectively. On the other hand the entire data are included in Appendix A.

Figure 6.2 Permanent Strain versus Time for Mix HL-8
 (@ T = 115 °C, σ = 165 kPa, D = 2.241 gm/cm³)

Figure 6.3 Permanent Strain versus Time for Mix HL-3
 (@ $T = 115\text{ }^{\circ}\text{C}$, $\sigma = 165\text{ kPa}$, $D = 2.227\text{ gm/cm}^3$)

In order to identify the general formula that can describe the relationship between the factors under study and the permanent strain, a preliminary analysis was carried out and presented in Appendix B. This analysis indicated that the effect of density, temperature, and stress on the permanent (irrecoverable) strain are as follows:

$$\epsilon_{irrecoverable} = c_1 t^{c_2} T^{c_3} e^{D_o c_4} \sigma \dots\dots\dots(6.4)$$

where

$\epsilon_{irrecoverable}$ = Irrecoverable strain

T = temperature ($^{\circ}\text{C}$)

D_o = density (gm/cm^3)

σ = stress (kPa)

t = time (sec)

C_1, C_2, C_3, C_4 = constants

To find the constants of the above equation, the linear regression analysis available in SPSS was applied on the data presented in Appendix A. The output results of this analysis indicated that C_1, C_2, C_3 , and C_4 for HL-8 were 0.01112, 0.218, 1.692 and – 5.795 respectively and R^2 was 0.92. On the other hand C_1, C_2, C_3 , and C_4 for HL-3 were found 0.01008, 0.208, 1.805 and –6.057 respectively and R^2 was 0.90.

Equations 6.5 and 6.6 illustrate the final models that describe the effect of density, temperature, and stress on the permanent strain of mix HL-8 and HL-3 respectively.

For the Mix HL-8,

$$\epsilon_{irrecoverable} = 0.01112 t^{0.218} T^{1.692} e^{-5.795 D_o} \sigma \dots\dots\dots(6.5)$$

$$R^2 = 0.92$$

For the Mix HL-3,

$$\epsilon_{irrecoverable} = 0.01008 t^{0.208} T^{1.805} e^{-6.057 D_o} \sigma \dots\dots\dots(6.6)$$

$$R^2 = 0.90$$

6.5. Modelling of Asphalt Concrete Compaction

Through the previous sections, the behaviour of asphalt concrete mixes under compaction conditions was identified by calculating the recoverable and irrecoverable strain components, which occurs during compaction. This section deals with using this behaviour in order to develop a compaction model that can predict the final density of the compacted mix as a function of initial density, compaction stress, compaction duration, and mix temperature. This model was developed as follows:

- Converting equations 6.5 and 6.6 to express the final density of mixes HL-8 and HL-3 instead of the irrecoverable strain.
- Correlating the results of the two models developed in the the previous step with the actual data measured during the field monitoring in order to calibrate the models results.

6.5.1. Compaction models

Using Equation 5.3 developed in the previous Chapter, Equations 6.5 and 6.6 were converted to express density instead of irrecoverable strain. So, by substituting for ϵ in Equation 5.3 from Equations 6.5 and 6.6, the compaction models for mix HL-8 and HL-3 can be written as follows:

For the Mix HL-8,

$$D = \frac{D_o}{1 - (0.01112 \, t^{0.218} \, T^{1.692} \, e^{-5.795 D_o} \, \sigma)} \dots\dots\dots (6.7)$$

For the Mix HL-3,

$$D = \frac{D_o}{1 - (0.01008 t^{0.208} T^{1.805} e^{-6.057 D_o} \sigma)} \dots\dots\dots (6.8)$$

Where

D = Final density (gm/cm³)

D_o = Initial density (gm/cm³)

6.5.2. Correlating Lab Results to Field Results

Although Equations 6.7 and 6.8 are used to predict the final density of the mix, it should be noted that these equations were developed based on data generated from the laboratory. Therefore, the equations have to be calibrated before being used in modelling the compaction process in the field. This calibration will be carried out by correlating the output results of these two equations with the actual data collected from the field. Tables 4.13 through 4.16 illustrate these conditions, for the first and second layers of mix HL-8, and for the layer of mix HL-3.

As indicated before, the asphalt concrete layer gained its density by applying a compaction load for a given number of passes, where the density increases with the increase in the number of passes. By reviewing the data of the densities included in Table 4.12, it can be seen that in some cases the measured density decreases with increasing the number of passes which is not logical. To be more specific, these cases occurred as follows:

- After pass number four (pneumatic) during compacting the layer of HL-3.
- After pass number five (pneumatic) during compacting the second layer of HL-8.
- After different passes during compacting the first layer of HL-8.

As a result, in the following part, the measured densities of the fifth and sixth compaction passes of the second layer of HL-8 as well as the measured densities of the fourth and fifth passes of layer HL-3 will be discarded from the analysis. On the other hand, it was not applicable to discard some of the density measurements of the first layer of mix HL-8. So, in this layer the regression analysis technique was applied to these measurements to find the best equation that can fit the available data. Then, this equation was used to estimate the density at each pass. Figure 6.4 shows the field density of the first layer of HL-8 as well as the best curve that can fit these data and physically satisfy the compaction condition (i.e. density increases with increasing the number of passes). Table 6.1 shows the field densities calculated using the Equation illustrated in Figure 6.4 for the first layer of HL-8 and the field densities of the other two layers after discarding the unlogical data.

Figure 6.4 Corrected Field Density of First Layer of HL-8

Table 6.1 Field Densities for HL-8 and HL-3 used for model calibration

Pass number	Field density (gm/cm ³)		
	layer # 1 (HL-8)	Layer # 2 (HL-8)	layer # 3 (HL-3)
1 (Steel roller)	2.294	2.277	2.285
2 (Steel roller)	2.323	2.344	2.324
3 (Steel roller)	2.341	2.358	2.332
4 (Steel roller)	2.353	--	--
1 (Pneumatic roller)	2.363	2.359	2.341
2 (Pneumatic roller)	2.371	2.365	2.347
3 (Pneumatic roller)	2.378	2.366	2.377
4 (Pneumatic roller)	2.383	2.380	---
5 (Pneumatic roller)	2.389	---	---
6 (Pneumatic roller)	2.393	---	---

Then, using Equations 6.7 and 6.8, the densities for the layers of HL-8 and the layer of HL-3 were calculated and illustrated in Table 6.2. The input data of Equations 6.7 and 6.8 were taken from Tables 4.13 through 4.16 and Table 6.1.

Table 6.2 Calculated Densities for HL-8 and HL-3

Pass number	Calculated density (gm/cm ³)		
	Layer # 1 (HL-8)	Layer # 2 (HL-8)	layer # 3 (HL-3)
1 (Vibratory roller)	2.204	2.187	2.197
2 (Vibratory roller)	2.349	2.335	2.336
3 (Vibratory roller)	2.372	2.390	2.365
4 (Vibratory roller)	2.383	--	--
1 (Pneumatic roller)	2.381	2.388	2.361
2 (Pneumatic roller)	2.387	2.386	2.365
3 (Pneumatic roller)	2.394	2.389	2.367
4 (Pneumatic roller)	2.399	2.388	---
5 (Pneumatic roller)	2.402	---	---
6 (Pneumatic roller)	2.406	---	---

The regression analysis was used to find the relationship between the calculated densities and field densities. By applying the regression analysis on the data of the three layers available in Table 6.1 and Table 6.2 the following equations were established.

For the first layer of Mix HL-8,

$$D_{corrected} = 0.4653D_{calculated} + 1.2572.....(6.9)$$

$$R^2=0.7731$$

For the second layer of Mix HL-8

$$D_{corrected} = 0.4389D_{calculated} + 1.3175 \dots \dots \dots (6.10)$$

$$R^2 = 0.9527$$

For the layer of Mix HL-3,

$$D_{corrected} = 0.3845D_{calculated} + 1.4376 \dots \dots \dots (6.11)$$

$$R^2 = 0.7273$$

Using the previous equations, the calculated densities shown in Table 6.2 were adjusted and presented in Tables 6.3, 6.4, 6.5 for the first and second layers of mix HL-8 and the layer of mix HL-3. As shown, the error between the field and the corrected densities did not exceed 1.25%.

Table 6.3 Field versus Corrected Densities for First Layer of Mix HL-8

Pass #	Field Density (gm/cm ³)	Corrected Density $D_{corrected}$ (gm/cm ³)	Error (%)
1-steel	2.294	2.283	-0.48
2-steel	2.323	2.350	1.16
3-steel	2.341	2.361	0.85
4-steel	2.353	2.366	0.55
1-pneumatic	2.363	2.365	0.08
2-pneumatic	2.371	2.368	-0.13
3-pneumatic	2.378	2.371	-0.29
4-pneumatic	2.383	2.373	-0.42
5-pneumatic	2.389	2.375	-0.59
6-pneumatic	2.393	2.377	-0.67

Table 6.4 Field versus Corrected Densities for Second Layer of Mix HL-8

Pass #	Field Density (gm/cm ³)	Corrected Density $D_{corrected}$ (gm/cm ³)	Error (%)
1-steel	2.277	2.277	0.00
2-steel	2.344	2.342	-0.09
3-steel	2.358	2.366	0.34
1-pneumatic	2.359	2.366	0.30
2-pneumatic	2.365	2.365	0.01
3-pneumatic	2.366	2.366	0.00
4-pneumatic	2.380	2.366	-0.59

Table 6.5 Field versus Corrected Densities for the Layer of Mix HL-3

Pass #	Field Density (gm/cm ³)	Corrected Density $D_{corrected}$ (gm/cm ³)	Error (%)
1-steel	2.285	2.282	-0.13
2-steel	2.324	2.336	0.52
3-steel	2.332	2.347	0.64
1-pneumatic	2.341	2.345	0.17
2-pneumatic	2.347	2.347	0.00
3-pneumatic	2.377	2.348	-1.22

6.6 Summary

This chapter dealt with the analysis of the output results of the experimental program as well as the field data collected during the monitoring process in order to model the compaction process of the constructed layers. The evaluation of the calculated densities indicated that the procedure suggested in this research work could be used in modelling the compaction process with high accuracy. The input parameters required for this modelling are preliminary density, mix temperature, compaction load, and loading time.

On the other hand, in the next Chapter an existing software that has the ability to take into consideration the viscous characteristics of the asphalt mix will be used to establish an alternative model for the compaction process. The advantage of such model over the previous one is the ability to take the characteristics of the supporting layers into consideration.

CHAPTER SEVEN
MODELLING THE COMPACTION PROCESS
USING EXISTING SOFTWARE (Model II)

7.1. Introduction

In this Chapter, an alternative compaction model was developed using existing software. The main advantage of this model over the one developed previously is its ability to take the characteristics of the supporting layers of the pavement cross-section into consideration. It should be noted that the experimental work associated with this model involves not only the asphalt layers, but also the supporting layers of the pavement cross-section. So, this chapter was organized as follows;

- Introducing the software that will be used in the compaction modelling and identifying the input parameters required for this software.
- Implementing an experimental program to determine the input parameters required for the software.
- Running the software and analyzing its output results in order to calculate the final density of the asphalt layers under study.
- Comparing the densities calculated in the previous step with field densities to evaluate the applicability of using the followed approach in compaction modelling.

7.2 Computer Software

KENLAYER computer program was developed mainly for analyzing flexible pavement and it was selected to be used in this research work. This program can be used to

determine the deformation developed in the pavement cross-section under a specific load (Huang, 1993). In this program, the viscous characteristics of the asphalt layer are represented by defining its creep compliance under the addressed conditions. The creep compliance at various times $D(t)$ is defined as follows:

$$D(t) = \frac{\varepsilon(t)}{\sigma} \dots\dots\dots(7.1)$$

where

$\varepsilon(t)$ = time-dependent strain under a constant stress σ

Also, this program can assume the asphalt layer(s) as an elastic material and neglect its viscous characteristics. Regarding the supporting granular layers, they are represented in KENLAYER as elastic materials.

Since this program can accept the characteristics of the asphalt layer as an elastic material and also can take the viscous characteristics into consideration, it can be used to determine the deformation occurring due to the viscous effect of the mix. This can be determined by running the software twice: the first run by assuming the asphalt layer as an elastic material and the second run by taking the viscous and elastic effects into consideration. The difference between the deformations occurring in the two cases represents the deformation resulting from the viscous effect. Then, this deformation can be converted, by Equation 5.3, to express the final density.

The input data required for this program are as follows;

- Supporting layers characteristics: Modulus of elasticity and Poisson's ratio
- Asphalt layers characteristics: Modulus of elasticity, Poisson's ratio, and creep compliance curve. But it should be noted that the creep compliance curves have to be constructed from the results of unconfined tests.

The next section deals with determining the input parameters required for the computer software.

7.3 Defining Supporting Layer Characteristics

As mentioned before the supporting layers of the pavement cross-section under study include; natural subgrade, 450-mm layer of granular B type II, then 150-mm layer of granular A. During the field monitoring, representative samples, from each layer, were taken to define their characteristics in the lab. The resilient modulus of the subgrade layer was defined based on the data included in Table 4.4. On the other hand the resilient modulus of the two granular layers (A and B type II) were defined by testing them in the lab.

7.3.1 Resilient Modulus of Subgrade

From the data included in Table 4.4, the soil classification according to AASHTO system is A-7-6. The California Bearing Ratio (CBR) of this type of soil is in average 7.5 (Garber and Hoel, 2002). And the relation between the resilient modulus (M_r) and the CBR for fine soil is defined as follows (Garber and Hoel, 2002);

$$Mr \text{ (psi)} = 1500 * CBR \dots\dots\dots(7.2)$$

So, by using the above equation, the Mr for the subgrade is in average 11250 psi (77.6 MPa). On the other hand the Poisson's ratio for the fine soil will be assumed 0.4 as reported by Huang (1993). It should be noted that this approach was selected due to the unavailability of the test equipment required to define Mr , for fine soil, directly in the lab.

7.3.2 Resilient Modulus of Granular Layers

In general, the resilient modulus test is carried out by applying a repeated vertical stress, in fixed cycles of load and release, on a cylindrical confined specimen. During the early cycles the plastic deformation will be accumulated and after that only the recoverable deformation will be recorded (see Figure 7.1). The resilient modulus is given by:

$$M_R = \frac{\sigma_d}{\epsilon_r} \dots\dots\dots(7.3)$$

$$\epsilon_r = \frac{\Delta H_r}{H} \dots\dots\dots(7.4)$$

where:

M_R = resilient modulus (MPa),

σ_d = deviator stress (MPa),

ϵ_r = resilient axial strain,

H = specimen height (mm), and

ΔH_r = recoverable vertical deformation (mm).

The resilient modulus tests were carried out at the National Research Council (NRC) laboratory. The specimens were prepared and tested according to the specifications of American Association of State Highway and Transportation Officials (AASHTO) under designation number T 292-91. The load sequences used in these tests are as illustrated in Table 7.1. The M_r tests for granular A were obtained by NRC and are presented in Table 7.2. The data included in this table were used to develop a relationship between M_r and the percentage of the water contents for granular A as shown in Figure 7.2.

Figure 7.1 Recoverable and Permanent Deformation during Resilient Modulus Test

Table 7.1 Load Sequence used in the Resilient Modulus Test

Load sequence	Deviator stress (kPa)	Confining stress (kPa)
LS1	48	103
LS2	52	103
LS3	69	103
LS4	83	69
LS5	35	69
LS6	48	69
LS7	69	69
LS8	83	35
LS9	35	35
LS10	48	35

Table 7.2 Resilient Modulus Result versus Water Content for Granular A

Water Content	Resilient Modulus (Mpa)	Density (gm/cm ³)
1.4	222.50	2.186
2.9	201.25	2.157
4.2	169.47	2.155
4.8	165.57	2.194
6	143.85	2.177
		Average = 2.174

Figure 7.2 Resilient Modulus Results versus Water Content for Granular A

As indicated in Figure 7.2 the relationship between M_r and water content for granular A can be written as follows;

$$M_r (\text{Mpa}) = 250.88 - 18.072 W_c \dots \dots \dots (7.5)$$

From the previous equation the M_r at 2.4% water content, which is the condition during compaction as indicated in Table 4.17, is 207.5 MPa.

On the other hand, the specimens of granular B type II required for M_r tests were prepared and tested with the help of the staff of NRC. The tested specimens were prepared at the same water content and density recorded in the field, which are 2.3 % and

2.196 gm/cm³, respectively (see Chapter 4). Figure 7.3 illustrates the compaction process of the tested specimens and Figure 7.4 illustrates one specimen during Mr test.

Figure 7.3 Compaction of Tested Specimen

Figure 7.4 Resilient Modulus Test

The output results of the tests indicated that the M_r of granular B type II is 658.9 MPa. Figure 7.5 illustrates a typical output result for one test sequence.

Poisson's ratio for the granular layers A and B type II was assumed 0.35 as reported by Huange (1993). Finally Table 7.3 summarizes the characteristics of the granular layers.

Figure 7.5 Typical Output of the Resilient Modulus Test

Table 7.3 Granular Layers Characteristics

Criterion	Granular A	Granular B type II
Mr (Mpa)	207.5	658.9
Poisson's ratio	0.35	0.35

7.3.3 Asphalt layers characteristics

a. Mix characteristics under compaction conditions

The behaviour of the asphalt concrete mix during compaction (recoverable and irrecoverable strains) was already investigated in the previous two chapters. In this investigation, the asphalt mixes were tested under confined effect. But it should be noted that, the characteristics of the mixes that will be entered to the software have to be obtained from unconfined tests.

The asphalt mix under compaction conditions is very loose, which can not be tested without the confined effect. Therefore, to estimate the characteristics of the mix as if it was tested without confining, the effect of the sand-confining medium on the test results was determined and then this effect on Equations 6.2, 6.3, 6.5, and 6.6 was removed as follows;

- Six specimens were prepared with the same density (2.304 gm/cm^3). The specimens were divided into two groups, three specimens each.
- The two groups were tested, using the creep device, under the same temperature (60°C), and stress (330 kPa). One group was tested without confining while the other one was tested under the confining effect of the sand, which was used previously in Chapter 5.
- The vertical strains occurred in the two groups were compared to estimate a factor that can show the effect of the sand-confining medium on the vertical strain.
- Then, applying this factor on Equations 6.2, 6.3, 6.5, and 6.6 to remove the effect of the sand-confining medium.

Figure 7.6 and 7.7 illustrate the output results of the confined and the unconfined tests respectively. Then, Figure 7.8 illustrates that the relationship between the results of the confined and unconfined test.

Figure 7.6 Strain of Confined Creep Test

Figure 7.7 Strain of Unconfined Creep Test

Figure 7.8 Relationship between the Strains of Confined and Unconfined Creep Test

Then, the factor estimated in Figure 7.8, which is 2.1151, was applied to Equations 6.2, 6.3, 6.5 and 6.6 to determine the mix behaviour without the sand-confining effect. Equations 7.6 to 7.9 illustrate the modulus of elasticity and the creep compliance of mix HL-8 and HL-3 without the confining effect.

- For HL-8

$$E(kPa) = 12847.58 e^{-0.0158T} e^{0.98D} \dots\dots\dots(7.6)$$

$$D(t)(m^2 / kN) = 0.0235 t^{0.218} T^{1.692} e^{-5.795D} \dots\dots\dots(7.7)$$

- For HL-3

$$E(kPa) = 19297.59 e^{-0.0165T} e^{0.871D} \dots\dots\dots(7.8)$$

$$D(t)(m^2 / kN) = 0.0213 t^{0.208} T^{1.805} e^{-6.057D} \dots\dots\dots(7.9)$$

where

E = modulus of elasticity

$D(t)$ = creep compliance

T = temperature ($^{\circ}\text{C}$)

D = density (gm/cm^3)

t = time (sec)

The Poisson's ratio of the two mixes is assumed to be 0.35 (Huang, 1993).

b. Mix characteristics after compaction

In addition to the characteristics of the mixes during compaction, the mix characteristics of layer HL-8 has to be determined for density 2.412 gm/cm^3 and temperature 25°C , which were the conditions of the first layer of HL-8 when the second layer was laid down. Also these were the conditions of the second layer of HL-8 when the layer of HL-3 was laid down. At such conditions the pavement layer will be considered elastic. To define the modulus of elasticity for mix HL-8 at such conditions, nine specimens were fabricated using Marshall compactor and 35 blows per specimen side. Then these specimens were tested using the creep device and under three stress levels (165, 330, and 495 kPa), three specimens per stress level. The results of these tests are illustrated in Figure 7.9. From this figure the modulus of elasticity for mix HL-8 under testing conditions is 1070 MPa.

Figure 7.9 Stress-strains Relationship for Mix HL-8 at 25°C

7.4 Compaction Model

The contact area between the load and the asphalt layer in KENLAYER has to be circular. Although this is not the real case during the compaction where the contact area in this case is a rectangle, it was found that using a circular contact area with a diameter equal to the width of the rectangular area will not significantly affect the deformation occurring in the asphalt layer. This is applicable as long as the thickness of the compacted layer is less than half the width of the rectangular area. So, to verify that assumption, a sensitivity test was carried out.

7.4.1 Sensitivity Analysis

During this test the pavement cross-section was assumed to be four layers. These layers include subgrade, granular layer A (450-mm), granular layer B type II (150-mm), and asphalt layer HL-8 (50-mm). The characteristics of the subgrade and the granular layers were taken as determined in Sections 7.3.1 and 7.3.2 and the characteristics of the asphalt layer HL-8 were calculated from Equations 7.6 and 7.7 at a temperature equal to 123 °C and a density equal to 2.323 gm/cm³. The applied stress and the contact time were assumed to be 318 kPa and 0.11, second respectively. Finally, the program was used to calculate the deformation occurring in the asphalt layer under six different circular contact areas. The radii of these areas were 12.5, 25, 37.5, 50, 62.5, and 75-mm. The output results of the program were summarised in Table 7.4. These results indicate that if the radius of the contact area is more than the thickness of the compacted layer, there is no effect on the deformation developed in the compacted layer.

Table 7.4 Vertical Deformation versus Contact Radius

Contact radius (mm)	Vertical deformation (mm)
12.5	0.4
25.0	0.5
37.5	0.6
50.0	0.6
62.5	0.6
75.0	0.6

7.4.2 Compaction Modelling

Using the results obtained in the previous sections of this chapter, all the compaction passes for the three constructed asphalt layers were simulated using KENLAYER software. The conditions of the asphalt layer during each compaction pass were obtained from the monitored data as presented in Chapter 4. Then, Equation 5.3 was used to calculate the densities from the output deformation obtained from the software. Tables 7.5 through 7.7 summarized the densities measured in the field and those determined from the output of KENLAYER software under the same conditions. The results of these tables illustrate that the error between the measured and the calculated densities did not exceed 4%.

Table 7.5 Software Calculated Density versus Field Density for First Layer of HL-8

Pass #	Field Density (gm/cm ³)	Software Calculated Density (gm/cm ³)	Error (%)
1-steel	2.294	2.203	-3.97
2-steel	2.323	2.353	1.29
3-steel	2.341	2.377	1.54
4-steel	2.353	2.389	1.53
1-pneumatic	2.363	2.379	0.68
2-pneumatic	2.371	2.387	0.67
3-pneumatic	2.378	2.393	0.63
4-pneumatic	2.383	2.399	0.67
5-pneumatic	2.389	2.404	0.63
6-pneumatic	2.393	2.408	0.63

Table 7.6 Software Calculated Density versus Field Density for Second Layer of HL-8

Pass #	Field Density (gm/cm ³)	Software Calculated Density (gm/cm ³)	Error (%)
1-steel	2.277	2.188	3.91
2-steel	2.344	2.351	-0.30
3-steel	2.358	2.396	-1.61
1-pneumatic	2.359	2.388	-1.23
2-pneumatic	2.365	2.386	-0.89
3-pneumatic	2.366	2.39	-1.01
4-pneumatic	2.38	2.389	-0.38

Table 7.7 Software Calculated Density versus Field Density for the Layer of HL-3

Pass #	Field Density (gm/cm ³)	Software Calculated Density (gm/cm ³)	Error (%)
1-steel	2.286	2.203	-3.63
2-steel	2.324	2.338	0.60
3-steel	2.332	2.369	1.59
1-pneumatic	2.341	2.361	0.85
2-pneumatic	2.347	2.365	0.77
3-pneumatic	2.377	2.368	-0.38

The data summarized in Tables 7.5 through 7.7 correlation equations were developed using regression analysis. These equations as are follows;

For first layer of Mix HL-8

$$D_{corrected} = 0.4529D_{calculated} + 1.2858.....(7.10)$$

$$R^2=0.7475$$

For second layer of Mix HL-8

$$D_{corrected} = 0.4376D_{calculated} + 1.3191.....(7.11)$$

$$R^2=0.9453$$

For layer of Mix HL-3

$$D_{corrected} = 0.3874D_{calculated} + 1.4303.....(7.12)$$

$$R^2=0.7129$$

The previous equations were used to adjust the calculated densities for the three layers. Tables 7.8 through 7.10 illustrate the densities after correction. The results in these tables indicate that the error between the adjusted and the measured densities did not exceed 1.23 %, which validate the followed procedure in modelling the compaction process.

Table 7.8 Corrected Density versus Field Density for First Layer of HL-8

Pass #	Field Density (gm/cm ³)	Calculated Density $D_{corrected}$ (gm/cm ³)	Error (%)
1-steel	2.294	2.284	-0.46
2-steel	2.323	2.351	1.23
3-steel	2.341	2.362	0.91
4-steel	2.353	2.368	0.63
1-pneumatic	2.363	2.363	0.01
2-pneumatic	2.371	2.367	-0.17
3-pneumatic	2.378	2.370	-0.35
4-pneumatic	2.383	2.372	-0.45
5-pneumatic	2.389	2.375	-0.60
6-pneumatic	2.393	2.376	-0.69

Table 7.9 Corrected Density versus Field Density for Second Layer of HL-8

Pass #	Field Density (gm/cm ³)	Calculated Density $D_{corrected}$ (gm/cm ³)	Error (%)
1-steel	2.277	2.277	-0.02
2-steel	2.344	2.348	0.17
3-steel	2.358	2.368	0.41
1-pneumatic	2.359	2.364	0.22
2-pneumatic	2.365	2.363	-0.08
3-pneumatic	2.366	2.365	-0.04
4-pneumatic	2.380	2.365	-0.65

Table 7.10 Corrected Density versus Field Density for the Layer of HL-3

Pass #	Field Density (gm/cm ³)	Calculated Density $D_{corrected}$ (gm/cm ³)	Error (%)
1-steel	2.286	2.284	-0.10
2-steel	2.324	2.336	0.52
3-steel	2.332	2.348	0.69
1-pneumatic	2.341	2.345	0.17
2-pneumatic	2.347	2.347	-0.02
3-pneumatic	2.377	2.348	-1.23

7.5 Summary

This chapter has presented an alternative method to model the compaction process using existing software. In this Chapter, the characteristics of the supporting layers of the monitored project were determined. Then these characteristics in conjunction with the behaviour of the asphalt concrete pavement were used to represent the characteristics of the pavement cross section during compaction in the software. The output results of the software were correlated with actual densities measured in the field to calibrate the model. The error between the field density and the calculated density did not exceed 1.23%.

CHAPTER EIGHT

SUMMARY, FINDINGS, AND RECOMMENDATIONS

The research work presented in this thesis dealt with the complex problem of asphalt compaction in the field. The main objectives of the research were: (1) to investigate field compaction of asphalt concrete mixes and define its limitations, (2) to review the consolidation theory and examine its applicability in modelling field compaction, (3) to explain the behaviour of asphalt concrete mix during compaction, (4) to carry out an experimental investigation of the behaviour of the mix during compaction, (5) to develop a model to predict the compaction quality as a function of the main variables, and (6) to verify the results through the design of a number field compaction tests.

The following section presents a summary of this research and the main findings, followed by the recommendations for practical implementation and future research work.

8.1 Summary

Field compaction of asphalt mixes is considered to be the most important operation before opening the road to traffic. Field compaction has also been recognized as the most significant factor that affects the long-term performance of asphalt concrete roads. However, the process of asphalt compaction has not been given sufficient scientific consideration by the paving industry. For example, there is no standard procedure or framework that can be used to predict the behaviour of the asphalt concrete mixes during compaction. The literature suggested that most of the research work in the area of asphalt

mix compaction has been limited to studying the effect of a specific factor(s) on the degree of the compaction. This approach led to establishing only general guidelines with very limited objectives and application ranges. Due to a lack of understanding of the behaviour of the mix during compaction, rolling test strips had to be used in the field prior to constructing the asphalt pavement. The purpose of these test strips is to define the best combination of the compaction variables that can be used in the project.

The research work presented in this thesis started with identifying the limitations associated with field compaction of asphalt concrete mixes and establishing the need to develop a scientific framework for asphalt compaction in the field. The next step aimed at understanding the mix behaviour during compaction through a review of different behaviours of engineering materials and the ingredient characterises of the asphalt mix. The following step focused on the design of a simple test procedure to determine the studied behaviour. Subsequently, a commercial paving project was selected to investigate the important compaction variables under actual field conditions. Thus, the mix behaviour was evaluated and compared under both field and laboratory conditions.

The results of both laboratory and field-testing were used to develop two compaction models. The developed models considered the significant variables affecting the compaction process. The models can be used to evaluate and assess the quality of compaction as a function of one or a combination of the input variables. The first model was based on correlating the results obtained from the laboratory testing to those monitored in the field. The second model involved using an existing computer software that allows using the output results of the suggested test procedure to represent the characteristics of the asphalt concrete mixes under compaction conditions. This model

has the extra advantage of modelling the asphalt multi-layer structure, including the effect of the underlying supporting layers.

8.2 Findings

The main findings of this research are outlined for four areas: (a) limitations of previous research work, (b) analytical investigation, (c) experimental investigation, and (4) analytical modelling.

Limitations of previous research work

The review of previous research work that has been carried out in the field of asphalt concrete pavements indicates several limitations. Previous research dealt with the effect of a specific factor(s) on the quality of compaction without paying significant attention to the behaviour of the mix itself during compaction. In addition, there is no standard procedure available that can be used to predict the behaviour of the mix under specific compaction conditions. As a result, the quality of the compacted mixes depends on the experience of the contractors and operators of the rolling equipment.

Clearly, since the entire operation was left to field operator experience, scientific progress in this area was limited to laboratory compaction and field trials. Furthermore, no compaction model was developed for predicting the final density of the compacted layer as a function of the main compaction variables. The absence of such a model necessitated the construction of test strips prior to the construction process. These test strips are used to define the appropriate combination of the compaction variables that

provided a final density meeting the specification limits. It should be noted that constructing these test strips involves a significant amount of money and time.

Analytical Investigation

A comprehensive study to describe the behaviour of the asphalt concrete mix during compaction was performed. The study included a review of the behaviour of engineering materials from the stress-strain-time point of view. More specifically, the characteristics of the mix ingredients (binder and aggregates) were considered. The results of this investigation showed that the viscous characteristics of asphalt mixes should be taken into account when studying the behaviour of the asphalt mix during the compaction process, especially at higher temperatures. The study also included examining the applicability of the consolidation theory of fine-grained soil in modelling asphalt concrete compaction. It was concluded that the equations of the consolidation theory cannot be used to model the compaction process of asphalt mixes. As an alternative approach, creep curves were found to be more suitable to investigate the mix behaviour during compaction. Subsequently, the development of the subsequent models was based on this finding.

Experimental investigation

Based on the behaviour of the asphalt concrete mixes during compaction, defined previously, a test procedure that can be used to predict this behaviour was established. An experimental program, including the main variables affecting the compaction process, was designed to determine the behaviour of two asphalt mixes in the lab (HL-3 and

HL-8). The actual behaviour of two mixes, (HL-3 and HL-8), was monitored during their compaction in the field.

Analytical Modelling

The main results of the experimental program were used to study the behaviour of the asphalt concrete mixes. This behaviour was determined by modelling the recoverable and irrecoverable strain components as a function of the compaction variables (compaction stress σ , time of loading t , and mix temperature T , density D_o). The developed equations were as follows,

For the Mix HL-8,

$$\varepsilon_{re\ cov\ erable} = 3.68 * 10^{-5} \sigma e^{0.0158T} e^{-0.98D_o}$$

$$\varepsilon_{irre\ cov\ erable} = 0.01112 t^{0.218} T^{1.692} e^{-5.795D_o} \sigma$$

For the Mix HL-3,

$$\varepsilon_{re\ cov\ erable} = 2.45 * 10^{-5} \sigma e^{0.0165T} e^{-0.871D_o}$$

$$\varepsilon_{irre\ cov\ erable} = 0.01008 t^{0.208} T^{1.805} e^{-6.057D_o} \sigma$$

The irrecoverable strain equations were used to model the compaction process by developing an equation that can relates the final density D to the initial density D_o , final density and irrecoverable strain $\epsilon_{irrecoverable}$. The compaction models for HL-8 and HL-3 are as follow,

For Mix the HL-8,

$$D = \frac{D_o}{1 - (0.01112 t^{0.218} T^{1.692} e^{-5.795 D_o} \sigma)}$$

For the Mix HL-3,

$$D = \frac{D_o}{1 - (0.01008 t^{0.208} T^{1.805} e^{-6.057 D_o} \sigma)}$$

In addition, existing software that has the ability to take into consideration viscous characteristics of the asphalt mix was used to establish an alternative model for the compaction process. The advantage of such a model over the previous one is its ability to take into account the characteristics of the supporting layers. The predicted densities using the suggested approaches indicated the applicability of modelling the compaction process based on the description of the mix behaviour presented in Chapter three and the test procedure established in Chapter five.

8.3 Recommendations

Based on the this research work, the following recommendations for practical implementation and areas for future research are offered:

1. Prior to the compaction in the field, the behaviour of the asphalt mix should be investigated in the lab under the expected range of the compaction conditions, and a compaction model should be developed for the layer(s) under construction. Then the optimum values of the compaction variables can be determined using the developed model(s).
2. As a result of the viscous characteristics of the mix during compaction, careful attention should be paid to increasing the contact time between the compaction equipment and the compacted layer. This will accelerate the compaction process and help complete the entire process before the mix temperature is decreased to undesirable level.
3. This research work focused on investigating the behaviour of two types of mix for a specific mix design (i.e. specific binder type, specific binder contents). Therefore, it is recommended to carry out future research that can incorporate the effect of the mix ingredients in the developed models. Availability of such a general model would facilitate more comprehensive analysis of the compaction process.
4. Another factor that was out of the scope of this research should be investigated. This factor is the maximum compaction load that can be applied on the mix without causing the shoving problem.
5. Effect of the compaction degree and the water content of the underlaying layers should be investigated for different pavement types: full depth asphalt concrete, asphalt concrete over emulsified asphalt base, and asphalt concrete over untreated aggregate base.

REFERENCES

- Abd El Halim, A.O., (1984), "The Myth of Reflection Cracking", Proc., *Canadian Technical Asphalt Association*, Vol. 39, Saskatoon, Saskatchewan, Canada, pp. 202-214.
- Abd El Halim, A.O., (1985), "Influence of Relative Rigidity on the Problem of Reflection Cracking", *Transportation Research Record* 1007, National Research Council, Washington, D.C., pp. 53-57.
- Abd El Halim, A.O., (1986), "Experimental and Field Investigation of the Influence of Relative Rigidity on the Problem of Reflection Cracking", *Transportation Research Record* 1060, National Research Council, Washington, D.C., pp. 88-98.
- Abd El Halim, A.O., Phang, W.A., and Haas, R.C., (1993), "Unwanted Legacy of Asphalt Pavement Compaction", *Journal of Transportation Engineering*, Vol. 119, No. 6, pp. 914-932.
- Abd El Halim, A.O., Haas, R., (1995), "Effect of Field Compaction Method on Fatigue Life of Asphalt Pavements", *Transportation Research Record* 1469, National Research Council, Washington, D.C., pp. 43-49.
- Abdel Nabi, R.M., Abd El Halim, A.O., Abd El Aleem, A.M., and El-Hussien, .M., (1995), "The use of New In-Situ Test Machine to Evaluate the Shear Strength of Asphalt Layers", Proceedings, *Annual Conference of Canadian Society for Civil Engineering*, Vol. 2, Ottawa, ON, pp. 89-98.

- American Society for Testing and Materials, (1994), "*Annual book of ASTM standards, Construction (Road and paving Materials; Paving Management Technologies)*", Section 4, Vol. 04.03, USA.
- Bell, C.A., Hicks, R.G., and Wilson, J. E., (1984), "Effect of Percent Compaction on Asphalt Mixtures Life", *Placement and Compaction of Asphalt Mixtures: ASTM STP 829*, F.T. Wanger, Ed., American Society for Testing and Materials, pp. 107-130.
- Bright, R., and Reynolds, E.T., (1962), "Effect of Mixing Temperature on Hardening of Asphaltic Binder in Hot Bituminous Concrete", *Highway Research Board, Bulletin 333*, National Research Council, Washington, D.C., pp. 20-38.
- Caterpillar web site, "www.caterpillar.com".
- Dickson, P.F., and Corlew, J.S., (1970), "Thermal Computations Related To The Study of Pavement Compaction Cessation Requirements", Proc., *Association of Asphalt Paving Technologists*, Vol. 39, pp. 377-403.
- Durham Geo Enterprises web site, "www.durhamgeo.com".
- Epps, J.A., and Monismith, C.L., (1969), "Influence of Mixture Variables on Flexural Fatigue Properties of Asphalt Concrete", Proc., *Association of Asphalt Paving Technologists*, Vol. 38, pp. 423.
- Federal Highway Administration (FHWA), (1995), "*Background of Superpave Asphalt Mixture Design and Analysis*".
- Finn, F.N., (1967), "Factors Involved in the Design of Asphaltic Pavement Surfaces", *Transportation Research Board, National Research Council*, NCHRP Report 39, Washington, D.C., pp.112.

- Finn, F.N., Epps, J.A., (1980), "Compaction of Hot Mix Asphalt Concrete", *Texas Transportation Institute*, Research Report 214-21.
- Garber, N.J., Hoel, L.A., (2002), "*Traffic and Highway Engineering*", Third Edition, Brooks/Cole Publishing Company, USA.
- Geller, M., (1982), "Compaction Equipment for Asphalt Mixtures", *Placement and Compaction of Asphalt Mixtures: ASTM STP 829*, F.T. Wanger, Ed., American Society for Testing and Materials, Philadelphia, Pa, pp. 28-47.
- Goode, J.F., and Owings, E.P., (1961), "Laboratory-Field Study of Hot Asphalt Concrete Wearing Course Mixtures", *public Roads*, Vol. 31, No. 11, pp. 221-228.
- Goodman, S., Hassan, Y., and Abd El Halim, A.O., (2000), "Permanent Deformation and the Development of an In-sit Shear Strength Device for Asphalt Pavements", *1st International World of Asphalt Pavement Conference*, Sydney, Australia.
- Huang, Y.H., (1993) "*Pavement Analysis and Design*", Prentice-Hall, Inc., New Jersey, USA.
- Hubbord, P., Gollomb, H., (1937), "The Hardening of Asphalt With Relation To Development of Cracks in Asphalt Pavements", Proc., *Association of Asphalt Paving Technologists*, Vol. 7, pp. 165-194.
- Hughes, C.S., (1977), "Vibratory Compaction of Bituminous Concrete-Where Does It Stand? ", Proc., *Association of Asphalt Paving Technologists*, Vol. 46, pp. 264-271.
- Hughes, C.S., (1989), "Compaction of Asphalt Pavement", *Transportation Research Board, National Research Council*, NCHRP 152, Washington, D.C.

- Kari, W.J., (1967) "Symposium, Compaction of Asphalt Concrete, Part B, Mix Properties As They Influence Compaction" Proc., *Association of Asphalt Paving Technologists*, Vol. 34, pp. 295-309.
- Lefebvre, J.A.A., (1965), "Effect of Compaction on the Density and Stability of Asphalt Paving Mixtures", Presented at 10th Annual Meeting of the *Canadian Technical Asphalt Association*.
- Lu, Y., and Wright, J.P., (2000), "Temperature Related Visco-Elasto-Plastic Properties of Asphalt Mixtures", *Journal of Transportation Engineering*, Vol. 126, No. 1, pp. 58-65.
- Marker, V., (1967) "Symposium: Compaction of Asphalt Concrete, Introduction" Proc., *Association of Asphalt Paving Technologists*, Vol. 36, pp. 288-294.
- Marker, V., (1972), "Symposium: Technology of Thick Lift Construction, Construction Methods", Proc., *Association of Asphalt Paving Technologists*, Vol. 41, pp. 354-364.
- Marshall, B.G., (1948)" Investigation of the Design and Control of Asphalt Paving Mixtures", *U.S. Army Waterway, Experiment Station*, Vols. 1 and 2, Vicksburg, Miss.
- McLeod, N.W., (1967), "Influence of Viscosity of Asphalt-Cement on Compaction of Paving Mixtures in the Field", *Highway Research Record* 158, National Research Council, Washington, D.C., pp. 76-115.
- Meyers, M.A., and Chamla, K.K., (1999), "*Mechanical Behaviour of Materials*", Prentice-Hall, Inc., New Jersey, USA.

- Noel, R. B., (1977), "Compacting Heavy Duty Highway Pavements", Proc., *Association of Asphalt Paving Technologists*, Vol, 46, pp. 309.
- Pagani, J., Hassan, Y., and Abd El Halim, A.O., (1999), "Compaction Technology – Then and Now", Proc., *Canadian Technical Asphalt Association*, Vol. 54, Quebec City, Quebec, Canada, pp. 27-48.
- Parker, C.F., and Hinman, W.E., (1960), "Steel-Tired Rollers", *Highway Research Board*, Bulletin 246, pp. 1-40.
- Pell, P.S., and Taylor, I.F., (1969) "Asphalt Road Materials in Fatigue", Proc., *Association of Asphalt Paving Technologists*, Vol. 38, pp. 371.
- Perl, M., Uzan, J., and Sides, A., (1983), "Visco-Elasto-Plastic Constitutive Law for a Bituminous Mixture Under Repeated Loading", *Transportation Research Record* 911, National Research Council, Washington, D.C., pp. 20-27.
- Rickards, I., Goodman, S., Abd El Halim, A.O., and Haas, R., (1998), "Consolidation – Fluid Flow: A New Way to Model Asphalt Compaction", Proc., *Canadian Technical Asphalt Association*, Vol. 53, Vancouver, BC, pp. 252-272.
- Rickards, I., Goodman, S., Pagani, J., Abd El Halim, A.O., and Haas, R., (1999), "Practical Realization of New Concept for Asphalt Compaction", *Transportation Research Record* 1654, National Research Council, Washington, D.C., pp. 27-35.
- Road Research Laboratory (RRL), (1952), " Soil Mechanics for Road Engineers", Her Majesty's Stationary Office, London, England.
- Santucci, L.E., and Schmidt, R.J., (1962), "Setting Rate of Asphalt Concrete", *Highway Research Board*, Bulletin 333, pp. 1-9.

- Scherocman, J.A. and Martenson, E.D., (1982), "Placement of Asphalt Concrete Mixtures", *Placement and Compaction of Asphalt Mixtures: ASTM STP 829*, F.T. Wanger, Ed., American Society for Testing and Materials, Philadelphia, Pa, pp. 3-7.
- Shames, I.H., and Cozzarelli, F. A., (1992) "*Elastic and Inelastic Stress Analysis*", Prentice-Hall, Inc., New Jersey, USA.
- Swanson, R.C., Nemec, J. Jr., and Tons, E., (1966) "Effect of Asphalt Viscosity on Compaction of Bituminous Concrete", *Highway Research Record* 117, pp. 23-53.
- Tons, E., and Krokosky, E.M., (1963), "Tensile Properties of Dense Graded Bituminous Concrete", Proc., *Association of Asphalt Paving Technologists*, Vol. 32, pp. 467.
- Transportation Association of Canada (TAC), (1995), "*Transportation in Canada: A statistical Overview*"
- Tunncliffe, D.G., Beaty, R.W., and Holt, E.H., (1974), "A History of Plants, Equipment and Methods in Bituminous Paving", Proc., *Association of Asphalt Paving Technologists*, Vol. 43A, pp. 222-296.
- US Army Corps of Engineers (USACE), (2000), "Hot-Mix Asphalt Paving Handbook"
- Yoder, E.J., and Witczak, M.W., (1975), "*Principles of Pavement Design*", Second Edition, Johan Wily & Sons, Inc.
- Younger, M. S., (1979), "Handbook for Linear Regression", Duxbury Press, Wadsworth, Inc., California, USA.

Zube, E., (1962), "Compaction Studies of Asphalt Concrete Pavement as Related to the Water Permeability Test", *Highway Research Board*, Bulletin 358, pp. 12-31.

APPENDICES

Appendix A

Permanent Strain Results For Mixes HL-8 and HL-3

**Figure A.1 Permanent Strain versus Time for Mix HL-8
(Group 1 @ $T = 115^\circ\text{C}$, $\sigma = 330\text{ kPa}$, $D = 1.980\text{ gm/cm}^3$)**

**Figure A.2 Permanent Strain versus Time for Mix HL-8
(Group 2 @ $T = 115^\circ\text{C}$, $\sigma = 165\text{ kPa}$, $D = 2.241\text{ gm/cm}^3$)**

**Figure A.3 Permanent Strain versus Time for Mix HL-8
(Group 3 @ T = 150 °C, σ = 330 kPa, D = 2.241 gm/cm³)**

**Figure A.4 Permanent Strain versus Time for Mix HL-8
(Group 4 @ T = 115 °C, σ = 495 kPa, D = 2.241 gm/cm³)**

**Figure A.5 Permanent Strain versus Time for Mix HL-8
(Group 5 @ $T = 115\text{ }^{\circ}\text{C}$, $\sigma = 330\text{ kPa}$, $D = 2.404\text{ gm/cm}^3$)**

**Figure A.6 Permanent Strain versus Time for Mix HL-8
(Group 6 @ $T = 115\text{ }^{\circ}\text{C}$, $\sigma = 330\text{ kPa}$, $D = 2.241\text{ gm/cm}^3$)**

**Figure A.7 Permanent Strain versus time for mix HL-8
(Group 7 @ T = 80 °C, σ = 330 kPa, D = 2.241 gm/cm³)**

Figure A.8 Permanent Strain versus Time for Mix HL-3
 (Group 1 @ $T = 115\text{ }^{\circ}\text{C}$, $\sigma = 330\text{ kPa}$, $D = 1.987\text{ gm/cm}^3$)

Figure A.9 Permanent Strain versus Time for Mix HL-3
 (Group 2 @ $T = 115\text{ }^{\circ}\text{C}$, $\sigma = 165\text{ kPa}$, $D = 2.227\text{ gm/cm}^3$)

Figure A.10 Permanent Strain versus Time for Mix HL-3

(Group 3 @ T = 150 °C, σ = 330 kPa, D = 2.227 gm/cm³)

Figure A.11 Permanent Strain versus Time for Mix HL-3

(Group 4 @ T = 115 °C, σ = 495 kPa, D = 2.227 gm/cm³)

**Figure A.12 Permanent Strain versus Time for Mix HL-3
(Group 5 @ T = 115 °C, σ = 330 kPa, D = 2.376 gm/cm³)**

**Figure A.13 Permanent Strain versus Time for Mix HL-3
(Group 6 @ T = 115 °C, σ = 330 kPa, D = 2.227 gm/cm³)**

Figure A.14 Permanent Strain versus Time for Mix HL-3

(Group 7 @ T = 80 °C, σ = 330 kPa, D = 2.227 gm/cm³)

Appendix B

Preliminary Analysis of Elastic and Plastic Strains Results for Mixes HL-8 and HL-3

B.1 Preliminary Analysis of Elastic Behaviour

As mentioned before, the objective of this analysis is to investigate the effect of each variable (temperature, stress, and density) separately on the elastic strain. In this part the average results for each group were used instead of the individual values. Tables B.1 and B.2 illustrate the average values of the data included in Tables 5.7 and 5.8 respectively.

Table B.1. Average Values of Elastic Strain Data for Mix HL-8

Group number	Temperature (°C)	Stress (kPa)	Density (gm/cm ³)	Average elastic strain (%)
1	115	165	2.264	0.45
2	80	330	2.267	0.47
3	115	330	2.293	0.79
4	115	495	2.342	1.02
5	115	330	2.416	0.71
6	115	330	2.194	0.83
7	150	330	2.342	1.32

Table B.2. Average Values of Elastic Strain Data for Mix HL-3

Group number	Temperature (°C)	Stress (kPa)	Density (gm/cm ³)	Average elastic strain (%)
1	115	165	2.271	0.40
2	80	330	2.278	0.42
3	115	330	2.306	0.72
4	115	495	2.353	1.01
5	115	330	2.444	0.64
6	115	330	2.205	0.76
7	150	330	2.351	1.24

Based on the data included in Tables B.1 and B.2, the effect of the density on the elastic strain for mixes HL-8 and HL-3 are represented in Figures B.1 and B.2 respectively. These figures were created using the experimental data of Groups 3, 5, and 6 for each mix. As indicated in these figures, the best curves that could fit the data of the two mixes are represented by exponential equations.

Figure B.1 Effect of Density on Elastic Strain of Mix HL-8

Figure B.2 Effect of Density on Elastic Strain of Mix HL-3

In order to study the effect of the other two variables (temperature and stress) on the elastic strain the effect of the density was neutralised firstly. These was carried out by assuming the density of the all groups as that one of Group 3, which are 2.293 for mix HL-8 and 2.306 for mix HL-3. Then adjusting the elastic strain for the all groups based on these new densities using the equations illustrated in Figures B.1 and B.2 for mix HL-8 and HL-3 respectively. As an example the adjusted elastic strain for Group 1 of mix HL-8 is calculated as follows;

$$\varepsilon_{adjusted} = \varepsilon \frac{e^{-0.7261*2.293}}{e^{-0.7261*density \text{ of Group1}}} = 0.45 \frac{e^{-1.665}}{e^{-0.7261*2.263}} = 0.44 \dots\dots\dots(B.1)$$

Tables B.3 and B.4 illustrate the modified elastic strain, based on the assumed new densities, for mixes HL-8 and HL-3 respectively. Using the data presented in Tables B.3 and B.4 the effect of temperature and stress on the elastic strain of mixes HL-8 and HL-3 were addressed. Figures B.3 and B.4 represent the effect of the temperature on the elastic strain for mixes HL-8 and HL-3 respectively. In this case the data of the Groups 2, 3, 5, 6, and 7 were used. It should be noted that for the two mixes the relationship between the elastic strain and temperature is exponential.

Table B.3. Modified Elastic Strain Data Mix HL-8

Group number	Temperature (°C)	Stress (kPa)	Actual Density (gm/cm ³)	Average elastic strain (%)	Base Density (gm/cm ³)	Modified elastic strain (%)
1	115	165	2.264	0.45	2.293	0.441
2	80	330	2.267	0.47		0.463
3	115	330	2.293	0.79		0.789
4	115	495	2.342	1.02		1.059
5	115	330	2.416	0.71		0.772
6	115	330	2.194	0.83		0.770
7	150	330	2.342	1.32		1.370

Table B.4. Modified Elastic Strain Data Mix HL-3

Group number	Temperature (°C)	Stress (kPa)	Actual Density (gm/cm ³)	Average elastic strain (%)	Base Density (gm/cm ³)	Calculated elastic strain (%)
1	115	165	2.271	0.40	2.306	0.393
2	80	330	2.278	0.42		0.411
3	115	330	2.306	0.72		0.722
4	115	495	2.353	1.01		1.027
5	115	330	2.444	0.64		0.685
6	115	330	2.205	0.76		0.726
7	150	330	2.351	1.24		1.264

Figure B.3. Effect of Temperature on Elastic Strain of Mix HL-8

Figure B.4. Effect of Temperature on Elastic Strain of Mix HL-3

On the other hand, Figures B.5 and B.6 show the effect of the stress on the elastic strain of mixes HL-8 and HL-3 respectively. These curves were created using the data of Groups 1, 3, 4, 5, and 6 for each mix. It should be noted that although R^2 for power equation was found higher than that of the linear equation, however the linear equation was selected. The reason for this selection refers to the computer software that will be used through the Chapter 8, where one of its requirements is a linear relationship between the stress and strain.

Figure B.5 Effect of Stress on Elastic Strain of Mix HL-8

Figure B.6 Effect of Stress on Elastic Strain of Mix HL-3

Now, by combining the effect of all variables (density, temperature, and stress) in one equation, the general formula to be used to determine the elastic strain as a function of density, temperature, and stress has to be as follows;

$$\varepsilon = C_1 \sigma e^{C_2 T} e^{DC_3} \dots \dots \dots (B.2)$$

Where

ε	= strain
$C_1, C_2, \text{ and } C_3$	= Constants
σ	= stress (kPa)
T	= Temperature ($^{\circ}\text{C}$)
D	= Density (gm/cm^3)

B.2 Preliminary Analysis of Permanent Strain

The objective of this analysis is to investigate the effect of each variable (temperature, stress, and density) separately on the permanent strain. The data included in Appendix A were used in this part.

Density Effect

Firstly the correlation factors between the test results carried out at different density was determined. To determine these factors for mix HL-8 the data of permanent strains illustrated on Figures A.1, A.5, and A.6 of Group 1, 5, and 6 respectively were used. The data of the permanent strain of Group 1 was plotted versus the permanent strains of Group 5 and Group 6 as illustrated on Figure B.7. From the equations illustrated on Figure B.7 it can be concluded that

- Correlation factor between the results of Group 5 and Group 1 is 0.3196.
- Correlation factor between the results of Group 6 and Group 1 is 0.0753.

Then these correlation factors were plotted versus the densities of Groups 1, 6, and 5, which are 1.98, 2.241, and 2.404 gm/cm³ respectively as shown in Figure B.8. From this figure it is found that the power equation could be used to describe the effect of the density on the permanent strain of Mix HL-8 with high coefficient of determination.

By applying the same procedure on the data of Mix HL-3, which are illustrated on Figures A.8, 12, and 13, the coefficient factors between the test results can be determined. Figure B.9 illustrates the coefficient factors and Figure B.10 illustrates the effect of density on the permanent strain results.

Figure B.7 Correlation Factors between Test Results of Mix HL-8 (Case of Density Change, Groups 1, 5, and 6)

Figure B.8 Effect of Density on the Permanent Strain of Mix HL-8

**Figure B.9 Correlation Factors between Test Results of Mix HL-3
(Case of Density Change, Group 1, 5, and 6)**

Figure B.10 Effect of Density on the Permanent Strain of Mix HL-3

Temperature Effect

The procedure followed previously was applied on the data of Group3, 6, and 7 of mixes HL-8 and HL-3 to investigate the effect of the temperature on the permanent strain. Figures B.11 and B.12 illustrate the correlation factors between the test results and the effect of the temperature on the test results respectively for mix HL-8. And Figures B.13 and B.14 illustrate the correlation factors between the test results and the effects of the temperature on the test results respectively for mix HL-3. From Figure B.12 and B.14 it was found that it is found that the exponential equation could be used to describe the effect of the density on the permanent strain of mixes HL-8 and HL-3 with high coefficient of determination.

Stress Effect

Using the data of Group2, 4, and 6 of mixes HL-8 and HL-3 the effect of the stress on the permanent strain was investigated. For mix HL-8, Figures B.15 and B.16 illustrate the correlation factors between the test results, and the effect of the stress on the test results respectively. And for mix HL-3, Figures B.17 and B.18 illustrate the correlation factors between the test results and the effects of the stress on the test results respectively. From Figure B.16 and B.18 it was found that it is found that the linear equation could be used to describe the effect of the density on the permanent strain of mixes HL-8 and HL-3 with high coefficient of determination.

**Figure B.11 Correlation Factors between Test Results of Mix HL-8
 (Case of Temperature Change, Groups 3, 6, and 7)**

Figure B.12 Effect of Temperature on the Permanent Strain of Mix HL-8

Figure B.13 Correlation Factors between Test Results of Mix HL-3 (Case of Temperature Change, Groups 3, 6, and 7)

Figure B.14 Effect of Temperature on the Permanent Strain of Mix HL-3

Figure B.15 Correlation Factors between Test Results of Mix HL-8 (Case of Stress Change, Groups 3, 6, and 7)

Figure B.16 Effect of Stress on the Permanent Strain of Mix HL-8

Figure B.17 Correlation Factors between Test Results of Mix HL-3 (Case of Stress Change, Groups 2, 4, and 6)

Figure B.18 Effect of Stress on the Permanent Strain of Mix HL-3

Finally, by combining the effect of all variables (density, temperature, and stress) in one equation, the general formula to be used to determine the permanent strain as a function of density, temperature, and stress has to be as follows;

$$\varepsilon_{irrecoverable} = c_1 t^{C_2} T^{C_3} e^{DC_4} \sigma \dots\dots\dots (B.3)$$

Where

$\varepsilon_{irrecoverable}$ = Irrecoverable strain

T = temperature ($^{\circ}\text{C}$)

D = density (gm/cm^3)

σ = stress (kPa)

C_1, C_2, C_3, C_4 = constants