Sand-Asphalt Mix Design with Available Materials for Hot and Arid Environments

By

Fathi MADWI

MANUSCRIPT-BASED THESIS PRESENTED TO ÉCOLE DE TECHNOLOGIE SUPÉRIEURE IN PARTIAL FULFILLMENT OF THE REQUIREMENTS FOR THE DEGREE OF DOCTOR OF PHILOSOPHY
Ph.D.

MONTREAL, April 16^{TH}, 2020

ÉCOLE DE TECHNOLOGIE SUPÉRIEURE
UNIVERSITÉ DU QUÉBEC

BOARD OF EXAMINERS

THIS THESIS HAS BEEN EVALUATED

BY THE FOLLOWING BOARD OF EXAMINERS

Mr. Gabriel ASSAF, Thesis Supervisor
Department of Construction Engineering at École de Technologie Supérieure

Mr. Kamal AL HADDAD, President of the Board of Examiners
Department of Electrical and Computer Engineering at École de Technologie Supérieure

Mr. Luc PELLECUER, Member of External Jury
Department of Civil and Environmental Engineering at University of the West of England (UWE Bristol)

Mr. Adnan SHIYAB, External Evaluator
Consultant engineer

Mr. Michel VAILLAN COURT, Member of Jury
Department of Construction Engineering at École de Technologie Supérieure

THIS THESIS WAS PRENSENTED AND DEFENDED

IN THE PRESENCE OF A BOARD OF EXAMINERS AND PUBLIC

ON APRIL 1TH, 2020

AT ÉCOLE DE TECHNOLOGIE SUPÉRIEURE

DEDICATION

Dedicated to:

To my beloved parents, Said Almadwi and Hadhom Almadwi,
and my all siblings, for their endless love and supporting
me spiritually throughout writing this thesis and my life in general…
Without your encouragements, nothing was possible.
To all of you, and for your unbounded support and love … I dedicate
this thesis.

ACKNOWLEDGMENT

I would like to express my deepest gratitude to my thesis supervisor, Prof. Gabriel J. Assaf, for accepting me as his Ph.D. student, and for his worthwhile supports and guidances. Under his supervision, it was a wonderful experience and opportunity to work hard to discover the world of the asphalt pavements.

I am also deeply thankful to the awesome laboratory technicians, Francis Bilodeau and Sylvain Bibeau. Their guidance helped me in all the time of lab work.

Moreover, I would like to thank my best friend Abu-Bakr Moussa, who helped me gathering the resources and literature materials.

I would especially like to thank Kamal Elhaddad, the president of my committee.

I also warmly thank Professor Luc Pellecuer, the members of the external jury.

A very special gratitude goes out to Professor Adnan Shiyab, the member of the external jury.

I would especially like to thank Professor Michel Vaillan Court, the members of the jury.

Special thanks to my colleagues for their endless assistance.

Mélange sables-bitumes adaptés aux climats chauds et arides

Fathi MADWI

RÉSUMÉ

Des techniques et méthodes spécifiques sont décrites dans cette thèse afin de développer des sables bitumes ayant en partie recours au sable du désert comme agrégat fin, lequel est disponible dans ces zones désertiques, le rendant rapidement accessible et peu coûteux.

Le défi demeure toutefois de rendre cet enrobé apte à assurer un bon comportement en comparaison avec un granulat concassé mécaniquement.

Il s'agit de produire des enrobés peu coûteux mais durables avec un recours en partie au sable désertique, et trouver un rapport admissible entre la teneur en sable naturel et la teneur en sable broyé et autres fillers, garantissant ainsi un sable bitume (SB) stable aux températures élevées.

En raison de ses propriétés viscoélastiques, le comportement rhéologique du SB est fortement affecté par les variations de la température et les autres conditions climatiques.

Les climats chauds et arides posent le défi des températures diurnes extrêmes, en particulier pour ce qui concerne les couches noires de la chaussée. Par ailleurs, le changement climatique a également un impact, notamment à cause des courants de vents chauds, dégradant davantage l'angularité du sable du désert. A ce titre, force est d'admettre que les changements climatiques peuvent remettre en cause les méthodes empiriques développées pour prédire le vieillissement de la chaussée, ce qui entraînerait une surestimation de la durée de vie prévue de la chaussée.

La détérioration des infrastructures routières requiert d'intervenir au moins sur la surface de la chaussée afin de préserver la sécurité et le confort des usagers. Ceci est d'autant plus vrai dans les régions chaudes et arides, à cause du fluage des enrobés sous les chaleurs extrêmes.

Quoique à partir des années 1970, il est devenu plus courant de réhabiliter les routes que de les reconstruire, il n'en demeure pas moins que des milliers de kilomètres sont encore construits chaque année dans les régions chaudes avec des budgets limités, ne permettant pas de recourir aux granulats de qualité de moins en moins disponibles et de plus en plus coûteux. C'est donc dans cette perspective que cette recherche devient utile pour les routes à faible volume (LVR), car elle permet de substituer, du moins en partie, le sable de carrière manufacturé et anguleux par du sable naturel et arrondi du désert.

Une validation est effectuée sur du sable du désert provenant du sud de la Libye, région qui fait partie du désert du Sahara et où les températures varient de 30 à 50 °C. Dans cette partie du monde, le sable est abondant mais rond. Les granulats de carrière et les sables de sablières

anguleux, propre aux enrobés de qualité, ne sont disponibles qu'à des centaines de kilomètreas, dans le nord du pays.

Afin de réduire ces coûts d'une part, et d'autre part afin de préserver les granulats de bonne qualité à d'autres fins, et enfin pour limiter la pollution associée au transport sur des longues distances, cette recherche démontre comment utiliser malgré tout le sable disponible dans le désert, au sud du pays.

Le sable naturel du désert ne répond généralement pas aux exigences d'angularité et de résistance à l'abrasion requise pour une utilisation en tant que matériau de chaussée non traité. Néanmoins, après traitement, il a été utilisé avec succès dans un certain nombre de projets routiers en Australie et ailleurs. Dans un enrobé conventionnel, le sable broyé mécaniquement fournit un grain anguleux qui résiste au compactage car les particules s'emboîtent naturellement les unes et les autres et forment un maillage continu et stable.

Le problème avec le sable d'origine naturelle, est qu'il a un grain de forme arrondi. Les routes construites à partir par le sable à grains arrondis développent des ornières prématurées car les particules ne s'emboîtent pas les unes dans les autres.

La démarche utilisée dans cette recherche permet d'obtenir des indications pour réduire la teneur en sable manufacturé dans la composition des sables bitumes. A cet effet, les essais utilisés dans cette étude sont l'essai Marshall, la presse à cisaillement giratoire, l'essai d'orniérage, l'essai de fluage, le module de résistance indirecte à la traction (ITSM) et le module complexe. Ils ont été effectués sur tous les échantillons afin de tirer des conclusions et des recommandations statistiques concluantes.

Les résultats démontrent que le choix du matériau affecte les propriétés volumétriques du sable bitume, ce qui affecte de manière significative son comportement. Ce mélange dépend de trois facteurs: la proportion de sable du désert par rapport au sable concassé mécaniquement; le choix du bitume; et le choix du filler minéral.

Dans cette recherche, le pourcentage optimal de sable du désert est de 33% pour tous les mélanges; ceci est complété par 63% de sable concassé et 4% de filler minéral (poudre de calcaire ou de brique). Ces échantillons ont été mélangés avec deux types de bitume différents: Performance Grade 70-10 (PG70-10) ou Liant de pénétration 60/70 (B60 / 70). L'effet du type de bitume sur le comportement viscoélastique du mélange a été étudié pour améliorer la durée de vie du mélange.

Les résultats des tests les plus déterminants sur la performance des sables bitume sont l'analyse du module complexe, le test de l'orniérage, le test ITSM et le test de fluage. Pour l'essai de module complexe, il a été constaté qu'à des températures élevées, caractéristiques des zones désertiques, la rigidité du mélange croît à mesure que le nombre de cycles augmente. En ce qui concerne le test d`orniérage, le mélange PG70-10 se révèle plus résistant à la déformation à l'orniérage que le mélange B60/70 couramment utilisé dans ces zones désertiques. Aussi, dans le même test, on observe que le mélange avec la poudre de brique comme filler était plus

résistant que le mélange avec un filler à base de calcaire. Les résultats de l'ITSM ont indiqué pour leur part que l'utilisation de la poudre de brique comme filler minéral au lieu de calcaire augmente le module de rigidité, la performance et la durabilité du sable bitume. L'essai de fluage a montré enfin que le mélange avec de la poudre de brique flue moins que le mélange avec de la poudre de calcaire. Cela signifie que le mélange avec de la poudre de brique déploie une meilleure résistance à la déformation en compression. En conclusion, les résultats de cette recherche contribuent à une meilleure compréhension des dosages des sables bitumes en milieu désertique.

Mots clés: Asphalte; Régions chaudes et arides; Performance; Sable-bitume; LVR, orniérage, module complexe, Superpave.

Sand-Asphalt Mix Design with Available Materials for Hot and Arid Environments

Fathi MADWI

ABSTRACT

Specific techniques and methods are described in this thesis in order to adapt Hot Mix Asphalt (HMA) so that desert sand can be used as a fine aggregate. In this way, the HMA can use a freely available local material. Such a locally available material is both accessible and low-cost but introduces the question of processing so that the final road construction is of comparable quality to that which is made using mechanically crushed aggregate. The challenge of this research is to meet the societal needs for low-cost HMA roads, made with readily available desert yet rounded sand, while finding an allowable ratio of natural sand to crushed sand and other fillers thereby ensuring a stable asphalt mix under hot temperatures.

Due to its viscoelastic properties, HMA is greatly affected by temperature variation, rate of loading and other climatic conditions. Hot and arid climates provide the challenge of extreme daytime temperatures, particularly for a black, road surface layer. Climate change is also considered to be worsening this effect, specifically because of hot wind currents further degrading desert sand, reducing its angularity. As such, changes in the climate may challenge conventional empirical methods used to predict pavement distress, resulting in an overestimation of projected pavement life.

Where road infrastructure is aging, pavement condition must be improved to maintain driver safety and comfort; this is a critical issue in hot and arid regions. Starting in the 1970s, it has become more common to rehabilitate road networks than to reconstruct them. For budgetary reasons, the growing costs of virgin aggregate, binders, and other materials have created a demand for more sustainable techniques that allow for the use of recycled or otherwise lower grade materials in HMA. Specifically, a sand-asphalt mix is a promising alternative to HMA, especially for Low Volume Roads (LVRs).

The laboratory investigations conducted in this research use round desert sand obtained from the desert region in the south of Libya. This region is part of the Sahara Desert where temperatures range from 30 to 50 degrees centigrade; there is an abundance of sand but there is no local source of what is typically the standard structural component for roadbuilding: an aggregate of gravel and sand. Currently, such aggregate must be transported from the north of Libya, hundreds of kilometers away. To reduce these costs, preserve good quality aggregates and reduce the transport associated pollution, this study develops techniques to use the sand readily available in the south. Typically, natural desert sand does not satisfy angular and abrasion requirements for use as a pavement material in its untreated state. Nonetheless, after processing, it has been used successfully in a number of road projects in Australia and elsewhere. In standard HMA construction, mechanically crushed sand provides an angular grain that resists compacting because the particles naturally interlock against each other and form a mesh. The problem with naturally occurring sand is that it has a rounded grain. Roads built from this rounded grain sand develop premature rutting because the particles do not interlock with each other. Although an HMA road using only sand as an aggregate will only ever be suitable for LVRs, a better understanding of how to integrate and process the naturally found sand will save a great deal of time and money. Beside the fact that desert sand is freely available, crushed sand is the by-product of the creation of larger aggregate; therefore it is the cheapest form of manufactured aggregate.

In this research, the HMA is mostly used as a surface layer. The tests used in this study are the Marshall, the gyratory compaction, the rutting analyzer, the creep, the Indirect Tensile Strength Modulus (ITSM), and the complex modulus. These tests have been performed on all samples in order to draw relevant statistical conclusions and recommendations.

The results demonstrate that the type of material affects the volumetric properties of the HMA that, in turn, significantly affects the rheological behaviour of the mix. This mix depends on three factors: ratio of desert sand to mechanically crushed sand; choice of bitumen; choice of mineral filler. In this research, the optimal percentage of desert sand has been found to be 33% for all mixes; this is complemented by 63% crushed sand and 4% mineral filler (either limestone or brick powder). These samples were mixed with two different types of bitumen, either Performance Grade 70-10 (PG70-10) or Penetration Binder 60/70 (B60/70). The effect of the bitumen type on the viscoelastic behaviour of the mix was investigated to improve the prediction of the service life of the mix and so was the type of filler (limestone or brick powder).

The most critical test results were the complex modulus analysis, the rutting test, the ITSM, and the creep tests. In the complex modulus test, it was found that the influence of high temperature conditions on the stiffness behaviour of the mix is higher as the number of cycles increases. Regarding the rutting test, the PG70-10 mixture was more resistant to rutting deformation than the B60/70 mix; in the same test, the brick powder mix was more resistant than the limestone mix. The ITSM results indicated that using brick powder as a mineral filler instead of limestone increases the stiffness modulus, performance, and durability of the asphalt surface mix. The creep test showed that the brick powder mixture has less creep than the mixture with limestone powder. This means that the brick powder mix increases the resistance to rutting deformation. The results from this research contribute to a better understanding of the sand-asphalt mix design method based on Superpave techniques.

Keywords: Asphalt mix; Hot-arid regions; Asphalt performance; LVRs, Rutting, Complex modulus, Superpave.

TABLE OF CONTENTS

Page

INTRODUCTION 13

CHAPTER 1 BACKGROUND AND LITERATURE REVIEW 17
1.1 Background 17
1.2 Asphalt Mixtures Varieties 18
1.2.1 Open Graded Mixtures 18
1.2.2 Dense Graded Mixtures 19
1.2.3 Gap-Graded Mixes 20
1.3 Aggregate Properties and Characteristics 20
1.3.1 Surface Shape and Texture 21
1.3.2 Gradation of Particle Sizes 21
1.3.3 Bitumen Absorption 21
1.3.4 Clay Content 22
1.4 HMA Design and Preparation 22
1.4.1 Marshall Method 22
1.4.2 Hveem Method 23
1.4.3 Superpave Method 24
1.4.4 LC Method of Mix Design 24
1.5 Flexible Asphalt Pavements 25
1.6 Asphalt Layers 26
1.7 HMA Formulation 27
1.8 Pavement Performance 28
1.9 Typical ACP Stresses 29
1.9.1 Stresses Induced by Axle Load 29
1.9.2 Climatic Stresses 30
1.9.3 Thermal Stresses 31
1.9.4 Bitumen Behaviour at High Temperatures 32
1.9.5 Viscoelastic Characteristics of HMA 33
1.10 Literature Review of Previous Applications of NDS in HMA 34

CHAPTER 2 RESEARCH FOCUS AND OBJECTIVES 38
2.1 Problem Statement 38
2.2 Research Objectives and Approach 39
2.3 Research Methodology 41
2.3.1 Introduction 41
2.3.2 Materials Ordering 41
2.3.3 Laboratory Tests for HMA Design 41
2.3.4 Determining the Optimal Ratio of Natural Desert Sand 42
2.3.5 Operational Framework 44
2.4 Research Significance 45
2.5 Outline of Thesis 45

2.6 Statement of originality....46

CHAPTER 3 EFFECTS OF ASPHALT BINDERS ON PAVEMENT MIXTURES USING AN OPTIMAL BALANCE OF DESERT SAND....48
3.1 Introduction....49
3.2 Background....49
3.3 MATERIALS AND TESTING PROCEDURES....54
3.3.1 Materials....54
3.3.2 Test procedure....57
3.3.2.1 Marshall Test....58
3.3.2.2 SUPERPAVE Gyratory Compaction (SGC) Test....58
3.3.2.3 Rutting analyzer Test....58
3.3.2.4 Complex modulus test....59
3.3.2.4.1 Linear viscoelastic (LVE) analysis....62
3.3.2.4.2 Linear Viscoelastic 2S2P1D Model....63
3.4 Result and discussion....65
3.4.1 Marshall test results....65
3.4.2 SGC Test Results....66
3.4.3 Rutting Analyzer Test Results....67
3.4.4 Complex Modulus Test Results....70
3.4.4.1 Complex Modulus E*....70
3.4.4.2 2S2P1D Model Formula....73
3.5 Conclusions....74
3.6 Further work....75

CHAPTER 4 EFFECTS OF NATURAL DESERT SAND ON PERMENET DEFORMATION BEHAVIOUR OF HOT MIX ASPHALT....76
4.1 Introduction....77
4.2 Background....77
4.3 Materials and Laboratory Testing Procedures....79
4.3.1 Materials....79
4.3.2 Sample preparation....82
4.3.3 Laboratory Testing Procedures....82
4.3.3.1 Index of Aggregate Particle Shape and Texture (IAPST) test....82
4.3.3.2 Marshall Test....84
4.3.3.3 Rutting Analyzer Test....85
4.3.3.4 Complex Modulus Test....85
4.4 Results of tests and discussion....87
4.4.1 Results of (IAPST) Test....87
4.4.2 Results of the Marshall Test....88
4.4.3 Results of the Rutting Analyzer Test....90
4.4.4 Results of the Complex Modulus Test....93
4.4.4.1 2S2P1D Model for the Linear Viscoelastic....93
4.5 Conclusions....99

4.6 Acknowledgments....100

CHAPTER 5 EFFECTS OF BRICK POWDER ON THE PROPERTIES OF HOT MIX ASPHALT....102
5.1 Introduction....103
5.2 Background....103
5.3 Materials and test procedures: 106
5.3.1 Materials....106
5.3.1.1 Aggregate....106
5.3.1.2 Filler....106
5.3.1.3 Asphalt binder....109
5.3.2 Mixture design and asphalt preparation:....109
5.3.3 Laboratory procedures:....110
5.3.3.1 French Rutting Tester (FRT)....110
5.3.3.2 Indirect Tensile Stiffness Modulus test (ITSM)....112
5.3.3.3 Moisture susceptibility test....113
5.3.3.4 Creep tests....115
5.3.3.4.1 Creep compliance....116
5.4 Results and discussion....118
5.4.1 Rutting results....118
5.4.2 ITSM results....120
5.4.3 Moisture susceptibility results....123
5.4.4 Creep compliance test....124
5.5 Conclusion....127
5.6 Acknowledgements....128

DISCUSSION AND CONCLUSION....130
DISCUSSION....130
CONCLUSION....130
RECOMMENDATIONS....132

APPENDIX I PERFORMANCE TESTING OF PAVING MIXES FOR LIBYA'S HOT AND ARID CONDITIONS, USING MARSHALL STABILITY AND SUPERPAVE GYRATORY COMPACTOR METHODS....134

APPENDIX II FINDING AN OPTIMAL BITUMEN AND NATURAL SAND BALANCE FOR HOT MIX ASPHALT CONCRETE IN HOT AND ARID REGIONS....151

APPENDIX III EDUCATION....167
APPENDIX IV PERSONAL PUBLICATION LIST....169
BIBLIOGRAPHY....171

LIST OF TABLES

Page

Table 1.1 Types of Hot-Mix Asphalt 17

Table 3. 1 Asphalt binder properties 55
Table 3. 2 Aggregate Properties 56
Table 3. 3 Asphalt mix composition and volumetric properties 56
Table 3. 4 Frequencies and number of cycles for temperatures tested 60
Table 3. 5 Effect of binder-type on stability values for marshall asphalt mixes 66
Table 3. 6 Summary of rutting analyzer test results 70
Table 3. 7 2S2P1D model and WLF constant parameters for the mixes 74

Table 4. 1 Asphalt binder properties 80
Table 4. 2 Gradation of fine aggregate 81
Table 4. 3 Moulds and tamping rods details according to (ASTM D3398, 2006b) 83
Table 4. 4 Crushed aggregates and natural sand properties 88
Table 4. 5 Marshall test results 89
Table 4. 6 2S2P1D Parameters 95

Table 5. 1 Physical properties of aggregates 106
Table 5. 2 Physical characteristics of the BP and LSP 107
Table 5. 3 Chemical characteristics of the BP and LSP 107
Table 5. 4 Drain down and specific surface test results 109
Table 5. 5 Physical properties of bitumen 109
Table 5. 6 Specifications for the FRT 111
Table 5. 7 BP and LSP results 120
Table 5. 8 SM at 40°C with a 124 millisecond rise time against cycle period 121
Table 5. 9 Effects of rise time on SM 121

Table AI. 1 Aggregate Properties 140
Table AI. 2 Marshall Stability Test Results 143

Table AI. 3 SUPERPAVE Gyratory Compactor Test Results....................................146

Table AI. 4 Rutting Analyzer Test Results 147

Table AII. 1 Coefficient of angle friction (ϕ) on the engineering properties of material 154

Table AII. 2 Aggregate Properties..155

Table AII. 3 Properties of Asphalt Binders and Temperatures 157

Table AII. 4 Fine Aggregate Grain Size Distribution 159

LIST OF FIGURES

Page

Figure 1. 1 Representative gradations for OGM 18
Figure 1. 2 Representations of gradations for DGM 19
Figure 1. 3 Standard gradations for GGM..20
Figure 1. 4 Cross section of a typical ACP 25
Figure 1. 5 Aggregate, bitumen, and HMA sample 27
Figure 1. 6 Characteristics of compacted HMA by mass and volume 28
Figure 1. 7 Factors influencing pavement performance...29
Figure 1. 8 Stress distribution at critical interface points in ACP 30
Figure 1. 9 How environmental forces affect pavement surface.................................31
Figure 1. 10 The behaviour of bitumen at high temperature...32

Figure 2. 1 Rut depth deformation after loading 43
Figure 2. 2 Flow diagram for laboratory analysis process 44

Figure 3. 1 Brookfield temperature vs viscosity curve for binders 55
Figure 3. 2 Aggregate gradation specifications for hot mix design 57
Figure 3. 3 Specimen preparation for the rutting tests 59
Figure 3. 4 Specimen preparation and strain measurement sectioning 60
Figure 3. 5 Various ranges of behaviour seen in the asphalt mix 63
Figure 3. 6 2S2P1D model 63
Figure 3. 7 SGC curves using PG 70-10 67
Figure 3. 8 SGC curves using B60/70...67
Figure 3. 9 Graphic of typical rutting test result of PG70-10 mixture 69
Figure 3. 10 Graphic of typical rutting test result of B60/70 mixture............................69
Figure 3. 11 Complex modulus results in Cole-Cole diagram......................................71
Figure 3. 12 Complex modulus results in Black Curve diagram 72
Figure 3. 13 Complex modulus results in master curve 73

Figure 4. 1 Gradation of crushed and natural sands 81
Figure 4. 2 Marshall testing machine and sample setup...84
Figure 4. 3 French laboratory slab compactor LPC 85
Figure 4. 4 MTS system and environmental chamber...86
Figure 4. 5 Test results for Marshall stability using two asphalt mixes 89
Figure 4. 6 Marshall flow test results for the two asphalt mixes.................................90
Figure 4. 7 Transverse rut profile (PG70-10 mix)...91
Figure 4. 8 Transverse rut profile (B60/70 mix) 91
Figure 4. 9 Rutting analyzer test on both mixtures PG70-10 and B60/70 93
Figure 4. 10 Cole-Cole Diagram 96
Figure 4. 11 Black Space diagram..97
Figure 4. 12 Module differences between S2 and S1 as a function of equivalent frequency...98
Figure 4. 13 Differences between phase angles as a function of the equivalent frequencies in S1 99

Figure 5. 1 The BP and LSP...108
Figure 5. 2 BP morphology under the SEM and XRF 108
Figure 5. 3 LSP morphology under the SEM and XRF 108
Figure 5. 4 Aggregates Gradation 110
Figure 5. 5 Scheme of rutting test (FRT) 111
Figure 5. 6 MTS measurement system and environmental chamber 113
Figure 5. 7 ITS measurement system and sample setup...114
Figure 5. 8 Schematic division of strain components as measured by the creep test.116
Figure 5. 9 Creep compliance test system 117
Figure 5. 10 BP and LSP rut test values on liner and logarithmic scales......................119
Figure 5. 11a Test values for mean rut depth at 60°C...120
Figure 5. 11b Comparison between BP and LSP mixture rutting test results at 60°C....120
Figure 5. 12 SM for HMA using BP or LSP 122
Figure 5. 13 ITS of various HMA mixes..123

Figure 5. 14 TSR values for various HMA mixes 123
Figure 5. 15 Creep Compliance Comparisons, 0°C 125
Figure 5. 16 Creep Compliance Comparisons, 20°C 125
Figure 5. 17 Creep Compliance Comparisons, 40°C 126

Figure AI. 1 PG Zone Map of Libya 138
Figure AI. 2 Grading of Libyan Natural Sand and Manufactured Crushed sand 140
Figure AI. 3 Brookfield Temperature vs Viscosity Curve for Asphalt Binders 141
Figure AI. 4 Marshal Stability 144
Figure AI. 5 Air Voids Percentage Va (%) 144
Figure AI. 6 Marshal Flow 145
Figure AI. 7 Superpave Gyratory Compactor (SGC) Test 146
Figure AI. 8 Rutting Analyzer Test 148

Figure AII. 1 Grain size distribution of fine aggregates 156
Figure AII. 2 Brookfield temperature vs viscosity curve for asphalt 156
Figure AII. 3 Specification of aggregate gradation for hot mix design 160
Figure AII. 4 Marshall stability results 162
Figure AII. 5 Superpave gyratory compactor results 163
Figure AII. 6 Wheel track results 163

LIST OF ABBREVIATIONS

AASHTO	American association of state highway and transportation officials
ASTM	American society for testing and materials
AC	Asphalt concrete
ACM	Asphalt concrete mix
ACP	Asphalt concrete pavements
APA	Asphalt pavement analyzer
BP	Brick powder
B60/70	Penetration grade
CEI	Construction energy index
DMA	Dynamic mechanical analyzer
DSP	Dune sand powder
DSR	The dynamic shear rheometer
E	Dynamic modulus
E^*	Complex modulus
ESG5	Called semi-grouted asphalt
FAA	Fine aggregate angularity
FRT	French rutting tester
FHWA	Federal highway administration of America
GDP	Gross domestic product
HMA	Hot mix asphalt
HMAC	Hot mix asphalt concrete

IAPST	Index of aggregate particle shape and texture
IDT	Indirect tensile test
ITS	Indirect tensile strength
ITSM	Indirect tensile strength modulus
LSP	Lime stone powder
LVE	Linear viscoelastic
LVRs	Low-volume roads
LTPP	Long term pavement performances
LPC	Laboratory slab compactor
M-E	Mechanistic-empirical
MEPDG	The mechanical-empirical pavement design approach
MTQ	Ministère des transports de Québec
NCHRP	National cooperative highway research program
OBC	Optimum bitumen content
PG	Performance grade
RCA	Recycled aggregate concrete
SEM	Scanning electron microscope
SGC	SUPERPAVE gyratory compaction
SHRP	The strategic highway research program
SIP	Stripping inflection point
SM	Stiffness modulus
SMA	Stone-matrix asphalt
SSA	Specific surface area

SUPERPAVE	Superior performing asphalt pavements
TSR	Tensile strength ratio
TTSP	The time-temperature superposition principle
V_A	Virgin aggregates
V_B	Percent bitumen volume
Va	Air voids content
VFA	Voids filled with asphalt
VMA	Voids mineral aggregate
XRF	X-ray fluorescence
UNDP	United nations development programme
2S2P1D	Two springs, two parabolic elements, and one dashpot

LIST OF SYMBOLS

N_{des}	Design number of gyrations
N_{ini}	Initial number of gyrations
N_{max}	Maximum number of gyrations
G_{mm}	Theoretical gravity max
G_{sb}	Bulk specific gravity
$V_{be\ (\%)}$	Effective binder volume
$B_{c\ (\%)}$	Bitumen content
φ	Phase angle
ε	Axial strain amplitude
σ	Axial stress amplitude
E^*	Norm of the complex modulus
σ_0	Is the max stress amplitude
ε_0	Is peak recoverable strain amplitude
ω	The pulsation, $\omega = 2\pi f$ (f is the frequency)
E_0	The static modulus when $\omega \to 0$
E_{00}	The glassy modulus when $\omega \to \infty$
k, h	Constant parameters such as $(0 < k < h < 1)$
δ	Constant
β	Parameter linked with η
η	Newtonian viscosity $[\eta\ (E_{00} - E_0)\beta_\tau$ when $\omega \to 0]$
τ_E	Characteristic time values
a_T	The shift factor at temperature T
T_{ref}	Reference temperature
C_1, C_2	Constant
V_{10}	Percent aggregate void once compacted at 10 drops for each layer
V_{50}	Percent aggregate void once compacted at 50 drops for each layer
M_{10}	Total average aggregate mass in the mould (g) once compacted at 10 drops for each layer
M_{50}	Total average aggregate mass in the mould (g) once compacted at 50 drops for each layer
S	Bulk specific gravity
V	Volume (ml) of cylindrical mould
I_a	Particle index
$R_{N+i/2}$	Area ratio
$N+i/2$	Load cycles
A_N^S and A_N^D	The areas of material in the shoulders and the depression respectively
A_{N+i}^S and A_{N+i}^D	Shoulder and depression asphalt mixture
F	Applied vertical load
L	Horizontal deformation obtained during the load cycle
H	Mean thickness of the cylindrical specimen
T_{cond}	Average indirect tensile strength of the wet specimens
T_{uncond}	Average indirect tensile strength of the dry specimens

P	Maximum test force
h	Height of test sample
d	Diameter of test sample
E	Norm of the modulus
ε_1	Vertical strain
ε_2	Horizontal strain
Δ	Percentage differences among the Modulus
$D(t)$	Time *t* creep compliance
GL	Gauge length (0.038 meters for 150 mm diameter samples)
D_{avg}	Average diameter of three samples
b_{avg}	Average thickness of three samples
P_{avg}	Creep load average
$\Delta X_{tm,t}$	mean of six normalized sample faces with horizontal deformations at time *t*
C_{cmpl}	Correction factor
$\frac{X}{Y}$	The ratio of the normalized horizontal trimmed mean as an absolute value

INTRODUCTION

0.1 Context of research

Asphalt began to be used for road construction with the advent of the automobile in the late 1800s. Since then, there have been great advances in asphalt road construction and modern techniques and equipment are very effective. Nevertheless, a road is only as good as the quality of the materials and the design and construction methods used. The equipment used for the construction cannot fully compensate for low-grade materials or insufficient methods.

The most commonly used material for the construction of roads worldwide is Hot Mix Asphalt (HMA). HMA consists of bitumen binders, mineral aggregates and fillers. The characteristics of these, and the nature of their interaction with each other, directly determine the nature of the final asphalt concrete pavement (ACP) and how it changes over its working life. Specific to road construction in desert regions, natural desert sand (NDS) and mechanically crushed sand (MCS) are evaluated for how they interact to create a mesh-like structure in the ACP (Kallas, B. F., Puzinauskas, V. P., & Krieger, 1962).

Five to ten percent of the mass of an HMA consists of a bitumen binder, either Performance Graded (PG) or a Penetration Binder (B). The other 90 to 95 percent of the total mass of the mix by weight is made up of aggregates. Aggregates can be defined as any mineral material that is chemically inert and that can be used in the mix in the form of fragments or particles within a determined size range. These sizes are divided into coarse (slag, screenings, gravel and crushed stone) and fine aggregate (MCS and NDS and mineral fillers). The qualities of the aggregate that affect the HMA include surface texture, shape and size of the particles, their composition, bulk density, specific gravity, voids, absorption, and porosity. The mesh created by the aggregate results in the road’s load supporting capacity. When there is too much natural sand, the asphalt is more prone to permanent deformation, such as rutting. These all contribute important aspects to the life and service quality of the ACP (Asi, 2007).

In hot and arid regions, such as in North African or Middle Eastern countries, roads often deteriorate more quickly than in temperate climates. These road surfaces suffer from poor mix designs resulting in substantial deformation (e.g., rutting, shoving), especially in desert regions. The binder now used in ACP for these roads is Bitumen 60/70 (B60/70). This binder is often produced in refineries such as those found in Libya, but it does not perform well in very hot conditions, resulting in substantial shear failure. Daytime air temperatures in the desert can be as high as 49 to 54°C, leading to road surface temperatures of over 70°C (Salem, Uzelac, & Matic, 2014). The Marshall test is useful for assessing material qualities under standard conditions, but it does not measure the fundamental rheological properties of the asphalt mix subject to these extreme temperatures (Almadwi & Assaf, 2017).

Southern Libya, like any area with arid sand deserts, has practical reasons for using local desert sands in asphalt mixes; nevertheless, too much desert sand aggregate in an ACP results in an ACP that is very likely to show stresses resulting in permanent deformation under use (Almadwi and Assaf, 2018). NDS particles become rounded through centuries of grinding against each other. Therefore, these sand particles are more rounded than comparable-sized manufactured and newly made aggregates. For practical purposes, the functional properties of the resulting ACP is also markedly different. The manufacturing of ACP mixtures with a high ratio of natural sand have attracted attention as a means of cost cutting and resource saving by efficiently using the existing natural options (Almadwi and Assaf, 2018).

0.2 Present Perspective

Road construction and maintenance is very costly both in terms of money as well as for the environment; as such, innovation is always important to reduce expenses and to minimize environmental impacts. This is particularly true when maintaining road networks in hot and arid countries where roads have comparatively poorer performance than elsewhere in the world. As such, both client satisfaction and productivity are of great concern (Australian Bureau of Statistics, 2012). Especially when it comes to low volume roads (LVRs) in remote areas, developing and refining new mix design techniques and production methods for ACP is key for cost-effective roads.

Even in regions with modern options to road travel, such as train travel, ~20% of shipments and ~90% of travel is still done on roads (Australian Bureau of Statistics, 2012). The construction of road infrastructure must take into account the political, social, and economic factors in any given country's recent history. ACP road network demands change with these political, social, and economic changes, leading to more reliance on systems of road infrastructure (Caeterling, J. S., Di Benedetto, C., Dorée, A., & Halman, 2012). In regions with LVRs, modern options to road travel and transport are often not practically possible for economic or other reasons.

CHAPTER 1

BACKGROUND AND LITERATURE REVIEW

1.1 Background

Hot Mix Asphalt (HMA) is the designation that is broadly used to cover a wide variety of different mixture types that are comprised of bitumen and aggregate; these necessarily are mixed at high temperatures. Typically, HMA is classified into three mix types: 1. open-graded, 2. dense-graded, and 3. gap-graded; these classes are determined by the aggregate gradation in the mix. See Table 1.1.

1- Open-graded mixes (OGM) are subsequently subdivided into 1.1 asphalt-treated permeable base and 1.2 open-graded friction courses.

2- Dense-graded mixes (DGM) are subdivided into: 2.1 conventional HMA, 2.2 large-stone, 2.3 sand asphalt, and 2.4 continuously graded mixes.

3- Gap-graded mixes (GGM) are subdivided into: 3.1 asphalt mixes with stone matrices and 3.2 concrete mixes with gap-graded asphalt. Civil engineers select an asphalt mix according to the project needs while at the same time accounting for project budget restrictions, aggregate availability and sourcing and bitumen binder availability (Federal Aviation Administration, 2013).

Table 1.1 Hot Mix Asphalt Varieties

Mix ID	**Mix Type**		
	1. OGM	**2. DGM**	**3. GGM**
Conventional Nominal	1.1 Bitumen-treated permeable base	2.1 Max aggregate particle size 12.5 - 19 mm	3.1 Stone Matrix Asphalt (SMA)
Large-stone Nominal	1.2 Porous friction layer	2.2 Max aggregate particle size from 25 - 37.5 mm	3.2 Conventional gap graded
Sand asphalt Nominal	-	2.3 Max aggregate particle size smaller than 9.5 mm	-

1.2 Asphalt Mixtures Varieties

1.2.1 Open Graded Mixtures

OGM is made up of aggregate particles with generally uniform grading and a bitumen binder. These mixes are used in drainage layers found at the surface of the asphalt concrete pavement (ACP) or in a structural layer. Two types of OGM are generally used. One is found in the surface layer, providing a draining surface to reduce tire splash and noise as well as to prevent vehicle hydroplaning. Such mixtures are usually referred to as open-graded friction course. Another type is called the bitumen-treated permeable base layer; this is made up of a uniform aggregate mix with greater maximum particle size tolerances than found in an open-graded friction course, i.e., 19 - 25 mm. This serves to drain water that has entered structural layers from subsurface or surface; see Figure 1.1 (Ongel, A., Harvey, J., & Kohler, 2007).

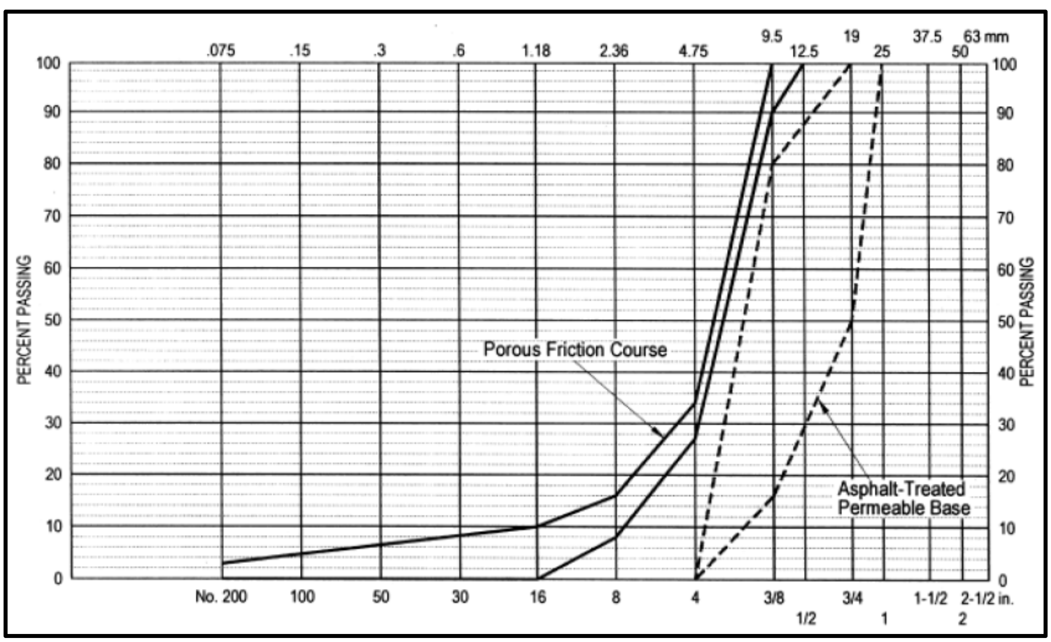


Figure 1. 1 Representative gradations for OGM Taken from (Ongel, A., Harvey, J., & Kohler, 2007)

1.2.2 Dense Graded Mixtures

DGM asphalt is made up of bitumen binders and continuously graded aggregate. Conventional HMA is made up of mixes with a designated upper aggregate particle size going from 12.5 to 19 mm. This aggregate comprises the greatest volume used in HMA worldwide. Large-stone aggregate mixtures are made up of coarse aggregate with a designated upper particle size of over 25 mm. Such mixes contain a greater percentage of coarse aggregate than do conventional mixes. Asphalt using sand as an aggregate is made up of aggregate particles that pass through a sieve with 9.5-mm holes. See Figure 1.2. Bitumen levels in such mixes are greater than with conventional HMA because there is a higher percentage of voids in the mineral aggregate (VMA) in the aggregate mix. If manufactured crushed sand (MCS) is not used in the mixture, this type of HMA shows very poor rut resistance (Ahmad, J., Rahman, M. A., & Hainin, 2011).

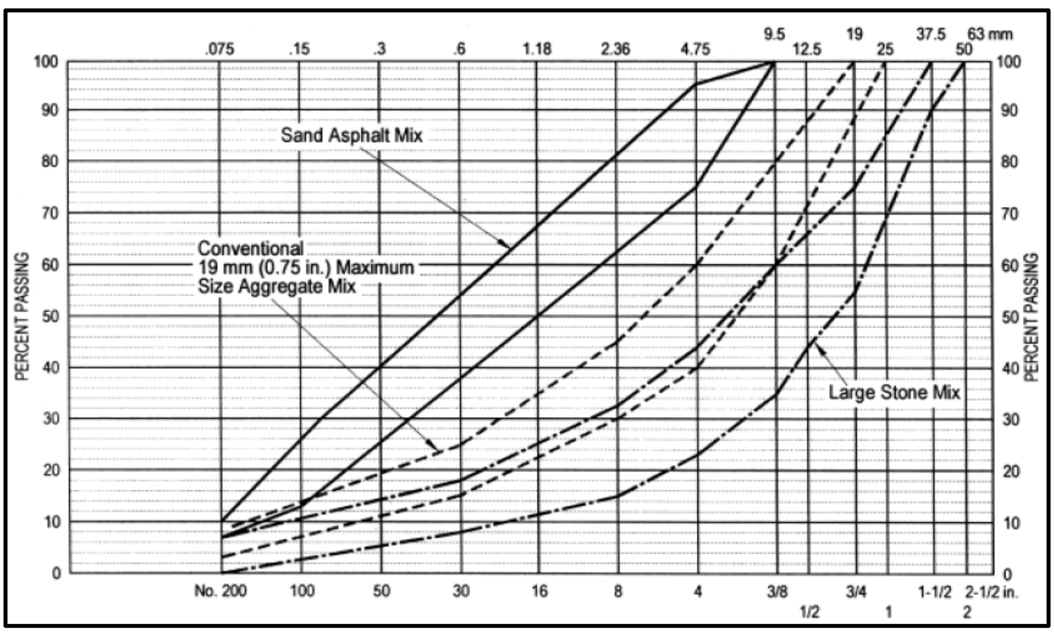


Figure 1. 2 Representations of gradations for DGM, Taken from (Ahmad, J., Rahman, M. A., & Hainin, 2011)

1.2.3 Gap-Graded Mixes

GGM has a function very much like that found in DGM. Both kinds of HMA are comprised of layers that are, when compacted according to international standards, largely impervious to water and very dense. Standard GGM has been used worldwide for decades. Aggregates sizes run from coarse to fine; intermediate particle sizes are often absent or only found in small quantities. The other GGM is called stone matrix asphalt (SMA). See Figure 1.3 (Federal Highway Administration, 2001).

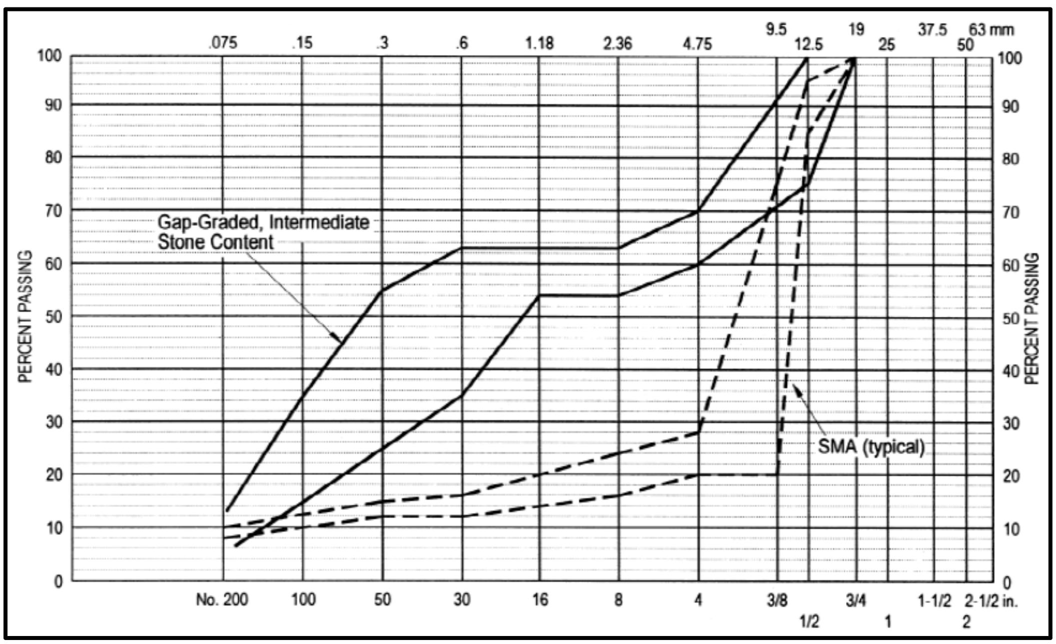


Figure 1. 3 Standard gradations for GGM. Taken from (Federal Highway Administration, 2001)

1.3 Aggregate Properties and Characteristics

Aggregate characteristics affect aggregate properties; this then affects the HMA performance. Such characteristics determine how much bitumen binder is needed to meet acceptable performance standards; likewise, this affects road construction (Transportation Research Board, 2011).

1.3.1 Surface Shape and Texture

The surface texture of an aggregate particle is a critical element that affects its coefficient of friction. This texture also shows a great influence on the final HMA's rutting resistance. Where the aggregate texture is rougher, the HMA demonstrates greater resistance to rutting. Over the period of construction, HMA with aggregate having a rougher texture will require more compaction work to get the needed HMA density than HMA with aggregate having a lower coefficient of friction. Aggregate shape also affects the mixture's rutting resistance; although needing more compaction at the time of construction, the finished ACP will have a greater working life and show greater rutting resistance over time. Angular aggregate has more rutting resistance because the edges of the MCS interlock, forming a flexible matrix; this contrasts with HMA made with natural desert sand (NDS) that has rounded edges and is less prone to interlocking (Transportation Research Board, 2011).

1.3.2 Gradation of Particle Sizes

The gradation of particle sizes in an aggregate mix is of critical importance. Where an aggregate mix has more than one range of maximum nominal particle sizes it can causes problems in the construction process because the different particle sizes will require different amounts of compaction. When the maximum nominal particle size is greater, with regard to layer thickness, the compaction takes more effort. Moreover, if the nominal maximum aggregate size exceeds one-third of the compacted thickness of the ACP layer, its surface texture is often changed, and the degree of density of the mix obtained by compaction may be reduced. To improve the resistance of HMA to rutting requires a greater maximum particle size and the proportion of coarse aggregate. (Federal Highway Administration, 2001).

1.3.3 Bitumen Absorption

How much bitumen the aggregate can absorb necessarily affects HMA properties. When aggregate particles absorb amount of bitumen, more bitumen must be used to compensate for what is absorbed to maintain an adequate film thickness surrounding the particles. Where such an adjustment has not been made, the resulting HMA is stiff and dry and needs more effort to compact according to industry standards; the resulting mix often ravels under axle-loading. Another problem is when an absorptive aggregate has already absorbed water; in this case, more time is needed in the HMA production so that the moisture can evaporate. If evaporation is not permitted, it will cause difficulties during the compaction process (Kandhal, P. S., & Khatri, 1991).

1.3.4 Clay Content

When fine aggregate passing a No. 4 sieve contains clay, it can affect the HMA's water sensitivity detrimentally. The minerals in clay coat the aggregate particles and interfere with the bonding process of the asphalt binders; this raises the likelihood of later water damage. To limit the amount of clay in the aggregate, the sand equivalent test is recommended (Lin, C. O. N. G., Xiaoguang, Z. H. E. N. G., & Weimin, 2006).

1.4 HMA Design and Preparation

To determine an optimal HMA, different sizes of aggregate are mixed with bitumen binders in lab testing. The mixes are then tested to find optimal bitumen percentages. Mix properties are a result of bitumen quantity and stiffness properties and how the aggregate responds to it. Optimum bitumen content (OBC) creates the best possible balance between stability (i.e., rutting resistance) and overall durability. Laboratory mix design uses either the Marshall method, the Hveem method, and the Superpave method (Highway Research Board, 1949), (Federal Aviation Administration, 2013).

1.4.1 Marshall Method

The U.S. Army Corps of Engineers (USACE) developed the Marshall method during World War II for paving runways for airplanes (Highway Research Board, 1949). The method is still widely used. The primary focuses of the Marshall technique are stability, flow, density, and void analysis. HMA strength is determined by the Marshall stability test according to ASTM D 1559 (2004) (American Society for Testing and Materials (ASTM), 2004); this is defined as the maximum axle load a compacted sample can tolerate at the testing standard of 60°C. Here, the sample was subjected to compressive axle loading at the rate of 50.8 mm/min until failure.

HMA flexibility is evaluated according to flow value; this is measured by the change in the sample's diameter under applied load direction. A dial gauge is attached to the sample to measure its plastic deformation, i.e., flow under load. At the point of failure, the plastic flow of a specimen is determined; this is considered the flow value. The volumetric properties of the mix is determined with a density-void analysis (Federal Aviation Administration, 2013).

1.4.2 Hveem Method

Working for the California Division of Highways, F. N. Hveem developed a method that has been used in California since the 1940s. Like with the Marshall technique, design details change from project to project. The equipment used to evaluate the mix is, nonetheless, largely the same. The principles behind this technique are:

1) the texture of the aggregate surface determines its stability;
2) OBC is a function of the aggregate's surface area, texture, and porosity, as well as the bitumen's stiffness;
3) where possible, OBC is designed to have a minimum of 4% Va to maintain sample stability and avoid bleeding.

The kneading compaction test (ASTM D1561) helps prepare samples for lab testing with a variety of OBC. Compaction was standardized to create densities representative of axle loads (Hveem, 1938).

1.4.3 Superpave Method

This technique is comprised of an initial volumetric design procedure followed by performance testing on the mix. The mix design adheres to the following steps:

1) Material selection;
2) Aggregate structure selection;
3) OBC;
4) Moisture susceptibility evaluation.

The component material selection necessitates choices with regard to the performance grade of the bitumen as well as an aggregate with qualities suitable for expected axle loads. Extremes of temperature at the road site largely determine the grade of bitumen needed. Aggregate is chosen according to needed or available aggregate angularity (including flat and elongated shapes) for coarse or fine aggregates; clay levels are also evaluated. For higher volume roads, the design requirements become stricter (Transportation Research Board, 2005).

1.4.4 LC Method of Mix Design

The method was developed by the Ministère des Transports du Québec and perfected at the Laboratoire des chaussées (hence its original name, “méthode LC”).
LC methods were adapted from both, France’s Laboratoire central des ponts et chaussées (LCPC) and the Superpave method, developed by the U.S. Strategic Highway Research Program (SHRP) methodologies. However, equipment and trial characteristics are directly related to the Superpave method alone. The use of the LC method results in mixes similar to those of Superpave, but generally with higher asphalt content. LC describes how performance is affected by the different properties of pavement mixtures, and how granulometry can be optimized for improved performance. It also summarizes the volumetric properties and provides a practical example of mix design (Ministère-des-Transports-de-Quebec, 2003).

1.5 Flexible Asphalt Pavements

ACP is a structure made up of different layers. The surface layer carries most of the axle load. This layer is the most expensive to build, consisting of a carefully designed mix of bitumen binders and aggregates. The surface layer is made up of a number of bonded sublayers. The function of this layer is to work as a flexible matrix, absorbing axle loads and transferring the load to underlying layers. Under the surface layer is the granular base course layer. This layer creates thickness and extends the working life of an ACP, protecting it against degradation from climatic and traffic conditions. Its thickness is typically from 100 to 300 mm. The base course layer is normally designed with coarse aggregate and therefore there is less bitumen. This layer reduces structural rutting deformation and keeps construction costs down. Recent research has indicated that the design of the base course layer is more essential than previously thought in terms of increasing the working life of the ACP; a well-designed base course layer will increase fatigue resistance and improve stability(G. W. Maupin and Diefenderfer, 2006); (Olard, 2012); (Theyse, H. L., De Beer, M., & Rust, 1996). Figure 1.4 illustrates a cross-section of a standard ACP.

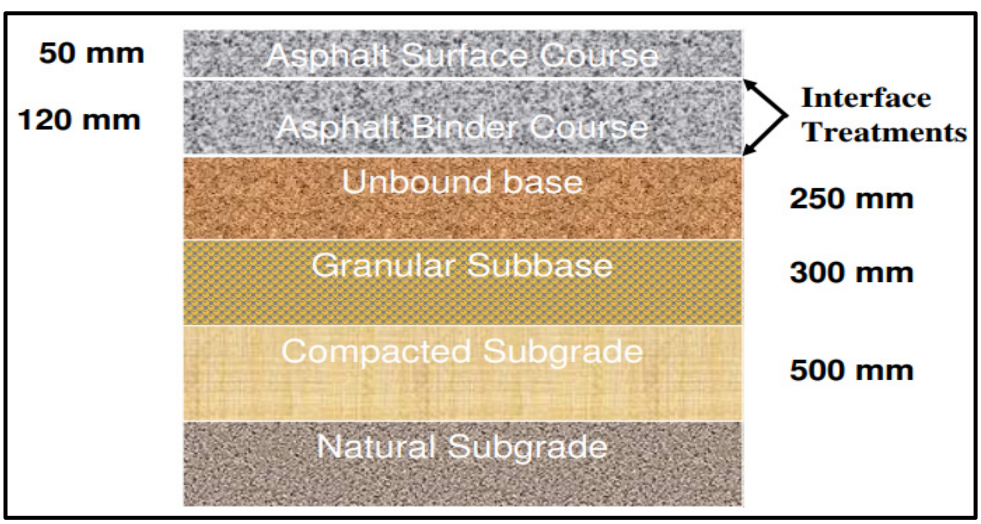


Figure 1. 4 Cross section of a typical ACP Taken from Jenks et al. (2011)

Thicknesses of ACP layers are designed based on expected axle-load distribution and volumes and adjusted for climatic conditions. To design the ACP, a variety of techniques have been developed. In NCHRP 1-37A, the mechanical-empirical pavement design (MEPDG) method was presented as a technique for the design and evaluation of ACP; MEPDG was developed from mechanistic-empirical (M-E) analyses which were based on material mechanics relating to various input data (e.g., climatic conditions, axle loads) to create an optimal pavement response characteristic. Based on the material and loading properties of the ACP as well as the expected climatic effects, the performance data from lab and field testing need to account for the calculated deflections, stresses, and strains; these structural responses are measured mechanically. Predictions of performance, including fatigue and rutting deformation, are viewed as structural responses and used as inputs for empirical testing (Yang H. Huang, 2004). The MEPDG model accuracy requires reliable field performance data to create a successful empirical model. The empirical models used for MEPDG can be used to predicted distresses directly; otherwise, distresses measured in the field can be used for calibration (Carvalho & Schwartz, 2006).

1.6 Asphalt Layers

Asphalt road surfaces create a complex structure that changes over time with varying levels of stresses from environmental and axle load conditions. The OBC and the various qualities of the aggregates, as well as the manufacturing methods all affect the HMA formulation. Figure 1.5 shows the standard properties of a HMA. Aggregate forms a matrix that has an elastic structure, but the nature of bitumen is viscoelastic; the responses of both are variable with time (Badeli, 2018).

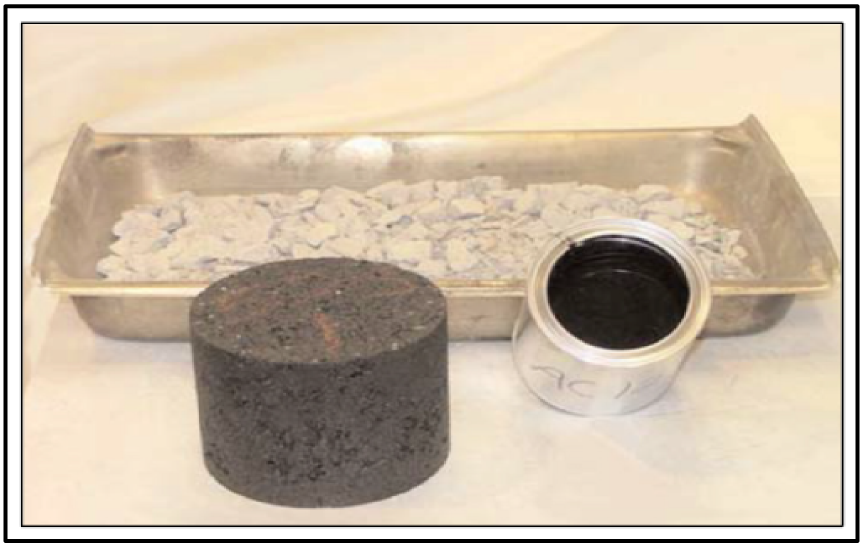


Figure 1. 5 Aggregate, bitumen, and HMA sample Taken from (Badeli, 2018)

1.7 HMA Formulation

ACP is primarily made up of bitumen and aggregate. Bitumen normally makes up approximately 5% of HMA by weight and aggregate makes up the other 95%. Figure 1.6 shows the principle parts of an HMA sample. In contrast, bitumen makes up approximately 10% of HMA by volume and aggregate makes up the other 85%; the remaining 5% is comprised of air voids. Fibers, anti-strip additives, crumb rubber, and mineral fillers are sometimes added in small amounts to HMA to improve working life and performance (European Asphalt Pavement Association, 2008).

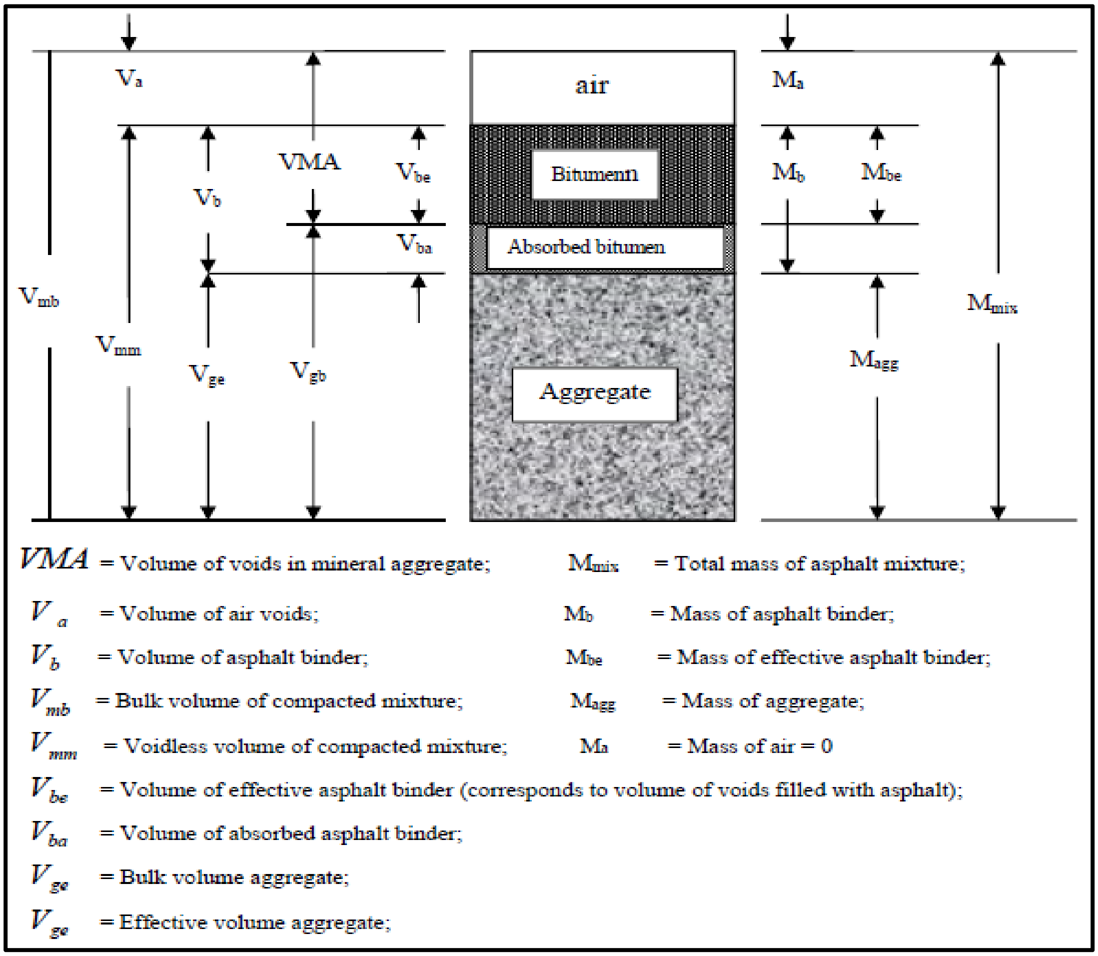


Figure 1. 6 Characteristics of compacted HMA by mass and volume
Taken from (Basueny, 2016)

1.8 Pavement Performance

Describing how the material properties of pavement responds to both climatic and axle loading conditions is summarized under the category of pavement performance. The guidelines pertaining to this are based on AASHTO 1993. This describes pavement performance according to how its performance changes over the course of its working life. It also takes into consideration how an ACP is able to function according to design. These factors can be described as how serviceability changes over the ACP's projected service life. Therefore, in the planning stages of a project, an engineer must not only account for the design and construction of the ACP but how to monitor and maintain the performance of the ACP and allow for rehabilitation as needed (Gupta Ankit, 2011). Figure 1.7 illustrates factors that affect ACP performance.

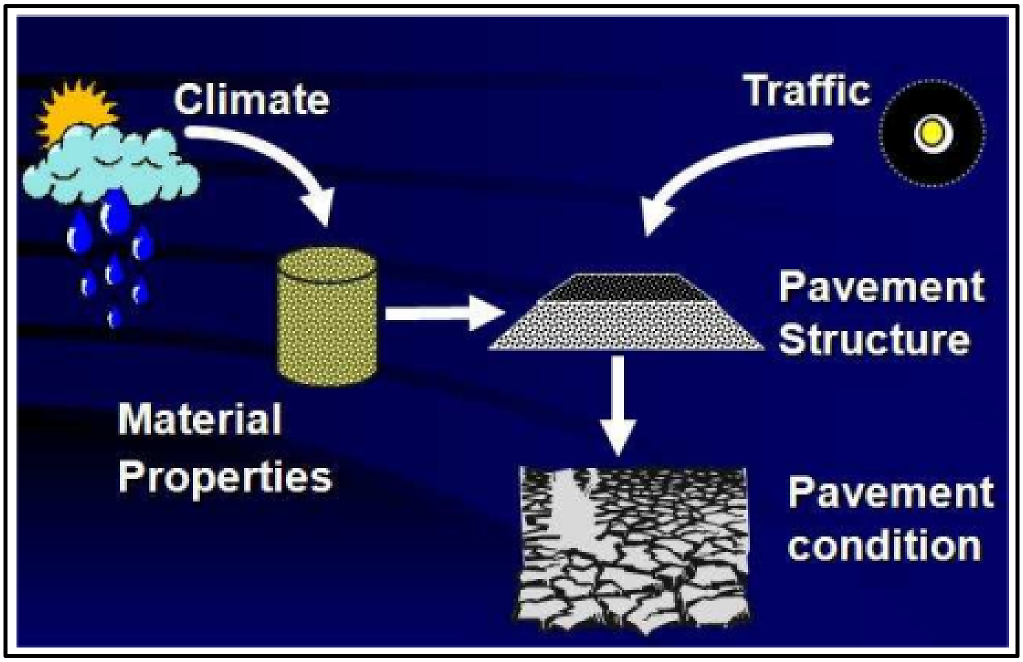


Figure 1. 7 Factors influencing pavement performance Taken from (Gupta Ankit, 2011)

1.9 Typical ACP Stresses

ACP is designed to undergo stresses. Civil engineers must have a profound understanding of how ACP stresses are caused and how to quantify them. ACP stresses can be defined as either axle load or climatically induced (Doré, 2009).

1.9.1 Stresses Induced by Axle Load

Axle loads are distributed through pavements in different ways, depending on whether the pavement is flexible, semi-rigid or rigid. Flexible ACP are designed to distribute the forces exerted on the surface through each layer. When a heavy vehicle runs over the ACP, the load force goes from the vehicle wheels through the ACP layers, creating compressive and tensile strains in multiple directions. This is shown in Figure 1.8. The principal stresses involved can be classified as follows: 1) compressive stresses coming from above, down through the asphalt and then the subgrade layers; 2) tensile stresses generated at the interface between the asphalt and subgrade layers. Fatigue cracking and permanent rutting deformation are produced by these stresses over years of service as the ACP undergoes heavy traffic loads (Doré, 2009).

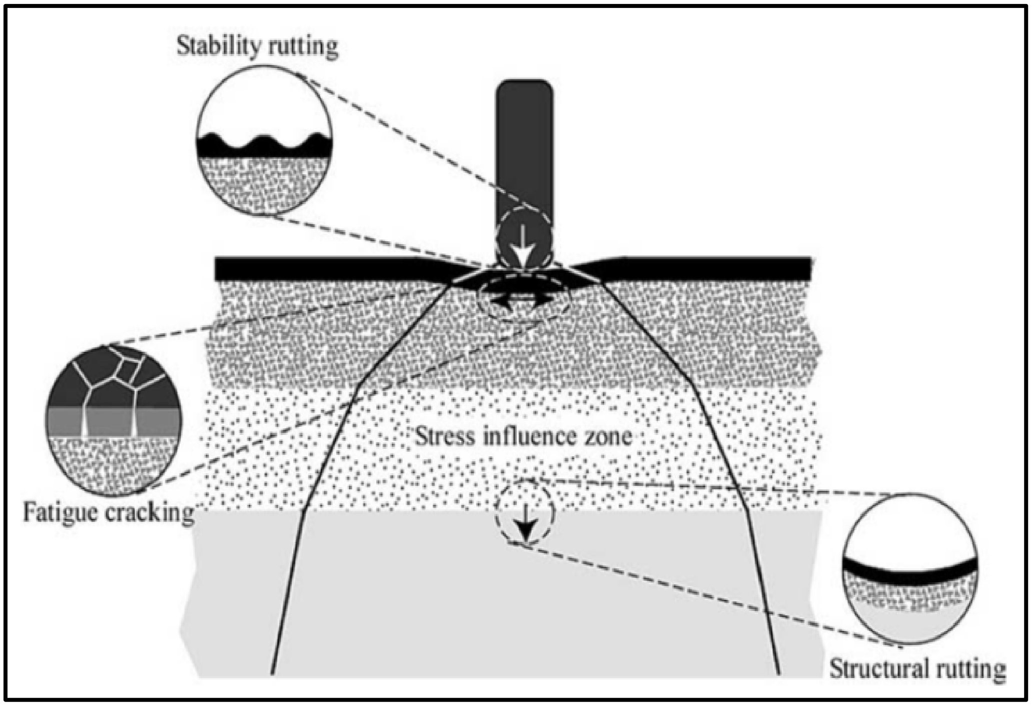


Figure 1. 8 Stress distribution at critical interface points in ACP Taken from (Badeli, 2018)

1.9.2 Climatic Stresses

Climatic stresses are created by the effects of humidity, precipitation, temperature, air pressure, wind, and solar heat. The principle environmental stresses in hot and arid regions are mostly from solar heat and hot desert winds. The environmental factors that have an effect on the HMA surface layer are shown in Figure 1.9. These factors have a variety of effects on HMA surfaces, including changes to the stiffness, and the loss of durability of the asphalt pavement. Cycles of high temperature result in a general diminishment of durability of asphalt mixes resulting in stripping distresses. HMA is also subject to other deformations and stresses from variability in thermal contraction and expansion, leading to cracking (Islam, M. R., & Tarefder, 2015).

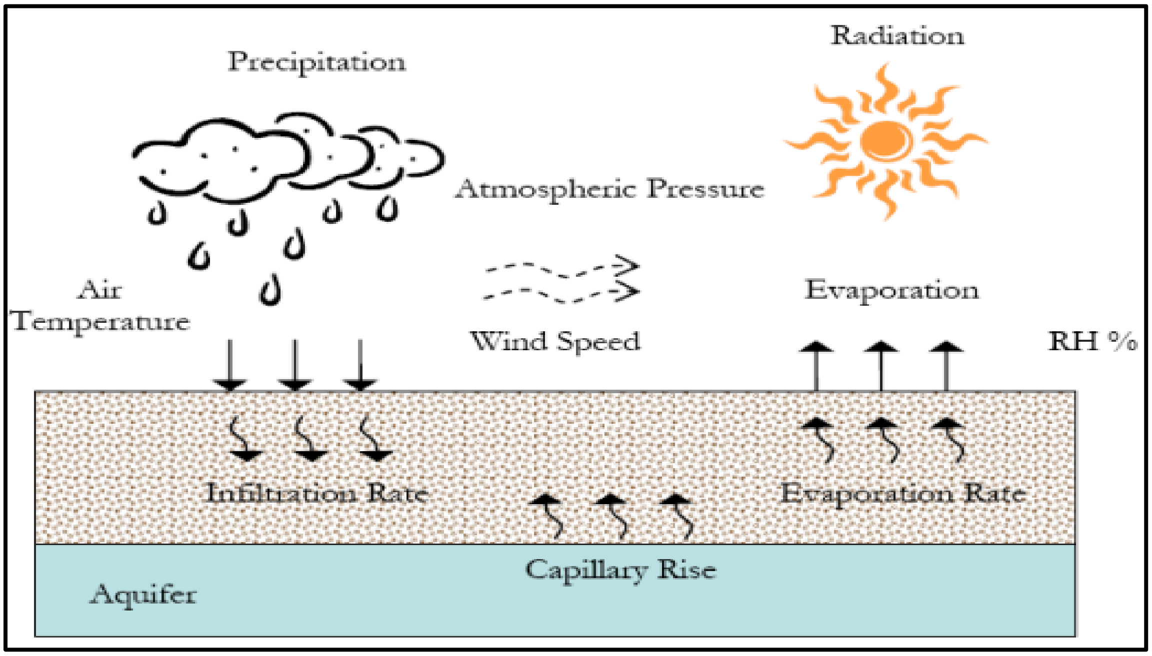


Figure 1. 9 How environmental forces affect pavement surface Taken from (Gupta Ankit, 2011)

Climactic factors are necessarily conditioned by seasonal changes. Basic factors to be accounted for in ACP design and maintenance are such things including seasonal changes pertaining to the resilient modulus in the ACP layers and the associated changes found in the subgrade resilient modulus. Seasonal variation in the different pavement layer moduli must be evaluated in order to account for variations of temperature and moisture the layers will be subjected to (Gupta Ankit, 2011).

1.9.3 Thermal Stresses

Thermal stresses develop slowly from daily cycles of heating and cooling (as well as other short-term variations in temperature); stress builds up between layers of pavement that are fixed and therefore cannot contract. These stresses do not grow in a linear way because asphalt has viscoelastic properties that allow for some release of stresses. Depending on the property of the bitumen binders, at particular temperatures, the asphalt begins behaving as a completely elastic material; at that point, thermal stresses rise according to temperature in a linear way (Doré, 2009).

1.9.4 Bitumen Behaviour at High Temperatures

In hot desert climates or when subjected to heavy traffic loading, bitumen behaves like a viscous liquid and it flows. The resistance of a liquid to flow is called viscosity. Viscous liquids that do not return to their previous position when they flow are called plastic. This is the case with bitumen in extremely hot climates; a small amount of the bitumen in the ACP flows when subjected to heat and repeated axle loads. The result of this is permanent rutting deformation. This rutting is also conditioned by the properties of the aggregate; as such, the asphalt mixture has plastic properties. See Figure 1.10 (Federal Highway Administration, 1998).

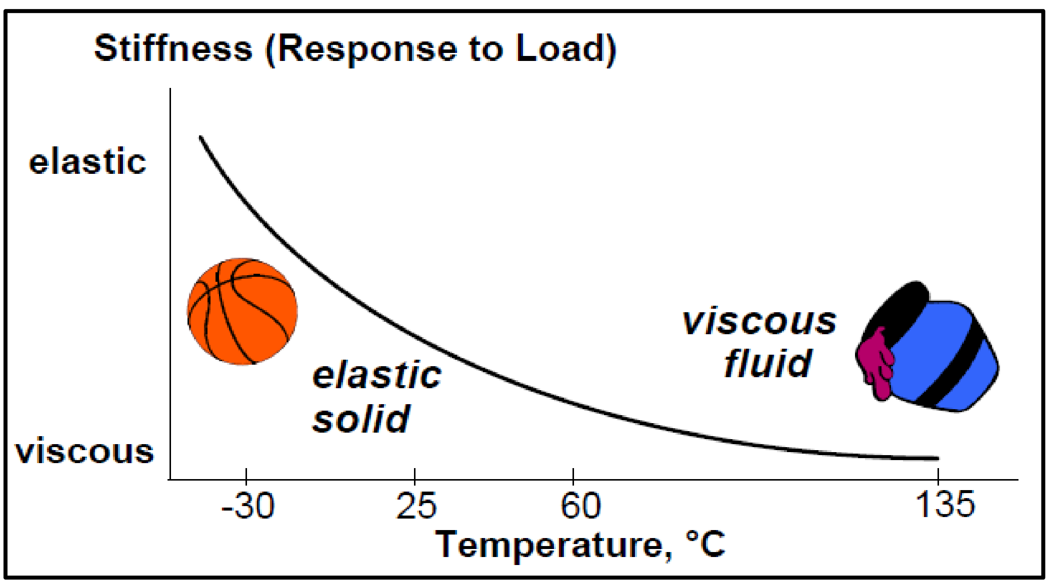


Figure 1. 10 The behaviour of bitumen at high temperature Taken from (Federal Highway Administration, 1998)

1.9.5 Viscoelastic Characteristics of HMA

As shown in Figure 1.11, bitumen mixtures can be classified according to strain amplitude and the number of load repetitions that it can withstand before failure. When subjected to a high strain amplitude for a limited quantity of cycles, the bitumen mix shows non-linear behaviour. This contrasts with bitumen subjected to several hundred cycles, each with a low level of strain; here the bitumen is linearly viscoelastic. Complex modulus tests are then carried out to evaluate an HMA's stiffness behaviour. Fatigue behaviour is exhibited when there are tens of thousands of cycles but the strain levels are low. Plastic deformation is possible when stress deviator cycles (starting from zero) are close to the rupture limit (Ramirez Cardona, D. A., Pouget, S., Di Benedetto, H., & Olard, 2015).

.

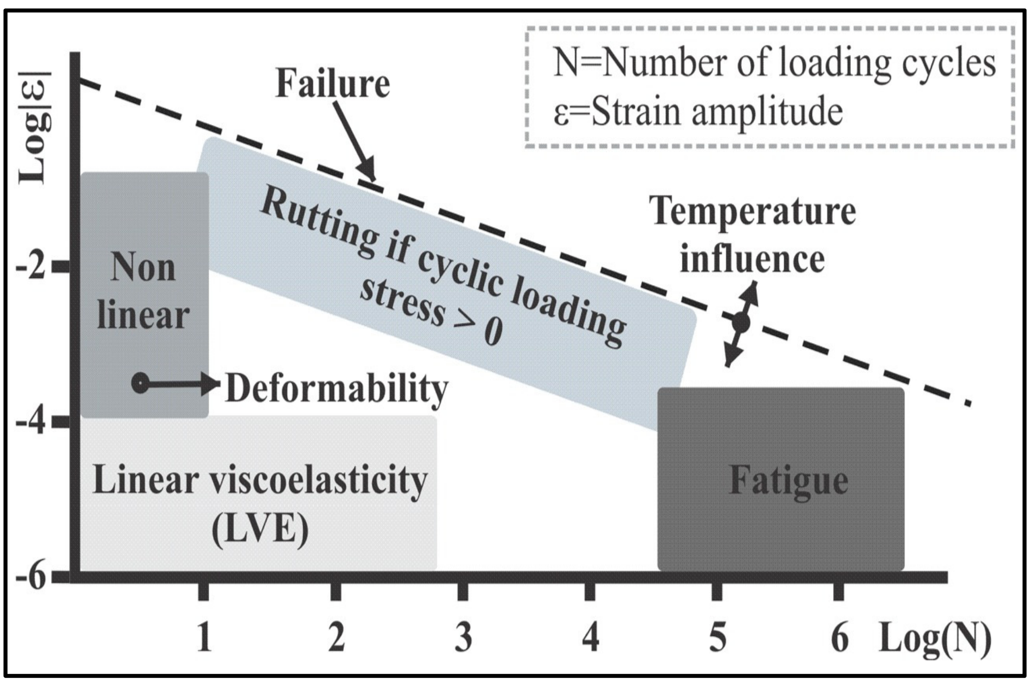


Figure 1. 11 Behaviour domains for HMA. Taken from (Ramirez Cardona, D. A., Pouget, S., Di Benedetto, H., & Olard, 2015)

1.10 Literature Review of Previous Applications of NDS in HMA

Due to the fact that an HMA is 95% aggregate by mass, the quality of the aggregate is of great importance. In a HMA, the aggregate may be made of either crushed gravel or stone. In both cases, the aggregate material must be consistently crushed; the particles should have a shape that is more cubical than elongated or flat. Furthermore, aggregates ideally should have minimal quantities of clay, dirt, dust, or other contaminants. The aggregate particles must withstand the axle loads in the finished ACP and, therefore, need to be resistant to abrasion and compression (Transportation Research Board, 2011).

Aggregate composition falls into types, according to size: fine aggregate, coarse aggregate, and mineral filler. Fine aggregates are defined as those that pass through a 2.36 mm sieve but cannot pass through a 0.075-mm sieve. Course aggregates cannot pass through the 2.36-mm sieve. Mineral filler is able to pass through the 0.075-mm sieve; it is a very fine material, sometimes called rock dust or mineral dust (European Asphalt Pavement Association, 2008).

Detailed lab research was carried out by the US Air Force Geotechnical Laboratory to find the optimum level of NDS and to determine its impact on the functional properties of HMA. The findings indicated that, as NDS increased, OBC decreased. NDS percentages also affected HMA stability; stability values decreased as NDS percentages increased. Mineral voids in the aggregates decreased as NDS percentage increased. The results from the lab testing generally showed that HMA that only used MCS was more resistant to permanent rutting deformation and that it had high levels of stability; this contrasted with HMA made from NDS. HMA with more than 20% NDS had a great tendency for permanent rutting deformation under heavy traffic loads (Ahlrich, 1991).

The effects of geometric shape, particularly particle angularity, on HMA performance was reported in (Lee, C. J., White, T. D., & West, 1999). The introduction of NDS was tested to see how it affected the mechanical behaviour of the HMA. There were two stages to this research. The first stage consisted of different HMA mixes for each of a number of fine aggregate blends. Blends of NDS and MCS were analyzed. The second stage of the study focused on improving the HMA designs that had demonstrated a great deal of permanent rutting deformation. The mixtures considered were a stone sand mix and a slag sand mix, both with S-shaped particle distributions. Strategies to improve the mixes included substituting some of the MCS with NDS, using mineral fillers, and adjusting the particle distributions in the aggregate blend. Adding NDS and mineral filler failed to improve the permanent rutting deformation of the MCS. In order to improve rutting performance the gradation had to be changed. Permanent rutting deformation was lowered with lower bitumen levels and higher relative mineral aggregate density.

Another study investigated the mechanical properties of HMA made from natural aggregates and recycled concrete aggregates (RCA) in ACP surface layers (Marques, V. D. C., de Queiroz, B. O., de Lacerda, D. M., Gouveia, A. M. D. A., & de Melo, 2014). The OBC was determined by using the Marshall method; the resulting HMA was evaluated for its mechanical properties. The HMA samples were made up of natural aggregates; RCA was then added in the following percentages: 25%, 50%, 100%. The findings suggested that it is possible to replace natural aggregates with up to 25% RCA for HMA surface layers. This lowered project costs and the environmental impact of the roadworks.

Another study looked at the mechanical characteristics of HMA that used RCA in low volume roads (Mills-beale & You, 2010). The RCA was used instead of aggregate made from Michigan traprock in the following percentages: 25%, 35%, 50%, and 75%. The permanent rutting deformation was evaluated using the rutting analyzer test, tensile strength ratio (for moisture susceptibility), dynamic modulus, indirect tensile test, resilient modulus, and the construction energy index. The RCA mixes performed less well in terms of dynamic stiffness than the control mix; when RCA was increased, HMA stiffness decreased. As RCA levels decreased, the tensile strength ratio and the moisture susceptibility increased; the 75% RCA mix failed to meet acceptable standards. According to the compaction energy index, using RCA could save compaction energy. A small amount of RCA in HMA was suggested for low volume roads.

Another study on the use of NDS on fatigue cracking on ACP tested two fine aggregates, NDS and MCS (Ahmed, 2016). NDS was substituted for MCS in the following amount: 0%, 25%, 75%, and 100% (by weight). All the aggregate passed through sieve No.8 and was retained by sieve No.200. Only one bitumen binder was tested, B40/50. Tests indicated that the best levels of NDS in the HMA was either 0% or 25% by mass.

Another investigation looked at the effect on HMA by using natural river sand (NRS) containing contaminants (Niazi, Y., & Mohammadi, 2003). For all mixes, bitumen B60/70 were used. One type of coarse aggregate was mixed with different fine aggregates. These fine aggregates were classified according to their geometric form and surface texture. Three fine aggregate mixtures were prepared: 1) MCS; 2) NRS; 3) a blend of MCS and NRS. The Marshall Method was used but it was found that the method was insufficient to determine the maximum recommended percentage of NRS in HMA; as such, more research was recommended. The findings from the unconfined compressive strength tests showed that NRS, particularly when containing contaminants, resulted in a reduction of the bearing and energy absorption capacity of the HMA; it raised the possibility of permanent rutting deformation and bleeding in ACP surface layers.

A further study attempted to determine the material properties of NDS asphalt mixes and to determine the practicality of its use in roadworks projects (Caro, S., Sánchez, D. B., & Caicedo, 2015). The Dynamic Mechanical Analyzer (DMA) test was used to evaluate the linear viscoelastic characteristics of the material as well as how the mixtures deteriorated under dynamic loading. The findings showed high variability in material properties in the sand-asphalt mixes. Furthermore, the bitumen binder had low complex moduli, high penetration and phase angle values. Testing also showed that the compaction temperature strongly affected the resistance of the mix. There was as much as 92% difference in the complex modulus between the specimens compacted at room temperature and the ones compacted at 140ºC. The high variability and the mechanical properties of the mix made it unsuitable for high-volume roadworks projects. The results, however, suggested that it could suitable to stabilize base or subbase layers or possibly used for the surface layer in low-volume roads.

Overall, the studies under review did not examine the effect of adding NDS to HMA as an end objective; this is to say, they did not attempt to solve the problem presented by building low volume roads in desert regions where NDS is freely available. For example, there was no discussion of optimal levels of NDS or the best bitumen to use or using mineral additives. As such, there was no systematic attempt to find other suitable materials to make the HMA of sufficient quality for use. Therefore, it is necessary to carry out this research to fill a gap in the area of low volume roads in hot and arid climates to provide guidelines on the feasibility and practically of using NDS in HMA. The findings of this research will have a major impact on the use of locally available materials, helping to minimize the cost of ACP construction.

CHAPTER 2

RESEARCH FOCUS AND OBJECTIVES

2.1 Problem Statement

The performance of HMA in hot and arid regions is influenced by several mechanisms. The literature review indicates that there are only a few studies that look at how much natural sand can be used for HMA in hot and arid regions. This study focuses on exactly that problem and determines the optimal ratio of natural sand that can be used in an HMA without compromising the mechanical characteristics of the HMA.

Local available materials: The south of Libya is part of the Sahara Desert where the temperatures range from 30 to 50 degrees Centigrade; there is an abundance of sand but there is no local source of what is typically the standard structural component for roadbuilding: an aggregate of gravel and crushed sand. Currently, this must be transported from the north of Libya, hundreds of kilometers away. To reduce these costs, this study develops techniques to use the sand already available in the south. Typically, this rounded sand does not satisfy industry requirements for use as a pavement material in its untreated state. Nonetheless, it has been used successfully in a number of road projects in Australia and elsewhere after processing. In standard HMA construction, mechanically crushed sand provides an angular grain that resists compaction because the particles naturally interlock each other and form a mesh. The problem with naturally occurring sand is that it has a rounded grain. Roads built from this rounded grain sand develop premature rutting because the particles do not interlock with each other. Although an HMA road using only sand as an aggregate will only ever be suitable for LVRs, a better understanding of how to integrate and process the naturally found sand will save a great deal of time and money. Beside the fact that desert sand is freely available, crushed sand is the by-product of the creation of larger aggregate; therefore it is the cheapest form of manufactured aggregate. In Libya, there is a need to develop a design system for roads that would account for load, materials and climate to produce mixtures with more stability and durability. This study is required to evaluate the applicability of the SUPERPAVE mix design method based on local construction materials properties and climatic conditions. For reference, the HMA is mostly used in this research as a surface layer.

Effects on ACP in hot and arid regions: When an asphalt mix heats and cools, the bitumen expands and contracts noticeably, whereas the aggregate shows little sign of change. When the aggregate heats, the film of bitumen that covers the aggregate thins and dissipates. This is because the bitumen is less viscous at higher temperatures and, over the course of years of heating and the mechanical force of heavy axle loads, the bitumen effectively drains out of the asphalt layer towards the surface. This causes micro-cracks in ACP; this process happens faster at high temperatures. The ACP becomes more brittle, and the micro-cracks eventually join together, creating visible cracks and deformation.

Study of the mechanism: Failure of asphalt materials can affect the expected service life of ACP before it undergoes excessive pavement deformation. In hot and arid regions, air temperatures can reach averages of between 50 and 65 degrees centigrade; road surface temperatures can reach up to 70 degrees centigrade. Top to bottom rutting deformation can be predicted based on loss of ACP stability due to repeated axel loads. Environmental conditions cause another kind of loading where temperature fluctuations result in the expansion and contraction of the bitumen with regard to the aggregate. Due to the high temperatures, this environmental loading is significant and creates stresses that reduce the service life of the ACP.

2.2 Research Objectives and Approach

The demands for roads continues to increase. These higher traffic volumes and greater overall axle loads require higher quality materials that have superior mechanical properties. At the same time, the growing environmental crisis requires a reduction of raw materials in construction.

Furthermore, the rate of HMA deterioration on highways in hot and arid areas has increased. This deterioration has been caused by higher traffic volumes, higher axle loads, increasingly higher tire pressure, and poor ACP construction quality control of both materials and methods of execution, all leading to an overall decrease in HMA performance. Rutting deformation is one of the most common forms of deterioration in ACP.

1. The overarching goal of this research project is to increase the durability and performance of sand-asphalt mixtures.

2. The goal of this research project is to evaluate the impact of using natural desert sand in HMA and assess how this might affect stability, rutting and stiffness of the sand-asphalt mix at high temperatures.

3. The main objective of this research project is to determine the optimal ratio of natural sand to mechanically-crushed-sand in order to improve the rheological behavior of sand-asphalt mixes.

The specific objectives of this research project are:

- To investigate the effect of increasing ratios of natural to crushed sand on the mechanical properties of asphalt mixtures;
- To evaluate the effect of high temperature on the rheological behavior of HMA and how this results in rutting;
- To investigate the effects of bitumen binder types as a variable against the control of a determined fine aggregate mix;
- To study the effect of an optimal ratio of natural sand on Stability, Density, Flow, Voids in Mineral Aggregate (VMA), Air Voids Content (Va) and Voids Filled with Asphalt (VFA) in asphalt mixtures;
- To evaluate the effects of the mineral filler replacement on the durability of ACP in hot and arid regions;
- To evaluate the effect of filler type and characterization of fillers properties on HMA characteristics and stiffness modulus.

2.3 Research Methodology

2.3.1 Introduction

The main objective of this research project was to determine the optimal ratio of rounded natural desert sand (NDS) to mechanically-crushed-sand (MCS) in order to improve the rheological behavior of sand-asphalt mixes. To conduct the lab work, two compaction methods were used (Marshall and Superpave). Materials used for this study were aggregates (NDS and MCS), bituminous binder- performance grade (PG70-10) and penetration grade (B60/70), and fillers- brick powder (BP) and limestone powder (LSP). All materials were prepared in accordance to the standard specification for road works published by: American Society for Testing and Materials (ASTM), American Association of State Highway and Transportation Officials (AASHTO), and Ministère des Transports de Quebec (LC), as a guide line to attain the laboratory works and materials to fulfill the road works circumstances in hot and arid environments.

2.3.2 Materials Ordering

The materials used in this research were supplied from Libya which are NDS and bitumen penetration grade (B60/70). As well as, the performance grade bitumen (PG70-10) was supplied from Arizona, United State.

2.3.3 Laboratory Tests for HMA Design

Laboratory investigation has been done to study the behavior of sand-asphalt mixtures by using materials from the site and exposing them to the same weather conditions of temperature and traffic loads throughout the service life.

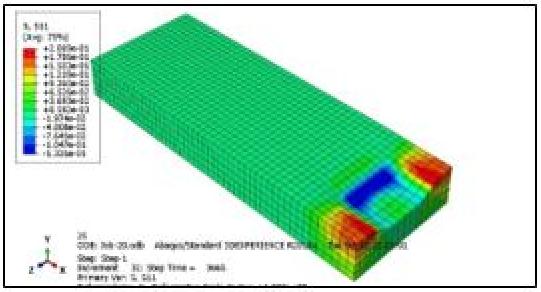
The laboratory work consisted of two series of tests with the first being tests done prior to mixing and second series being the tests done on prepared specimens. The tests conducted for the first series were sieve analysis, and determined of specific gravity for the aggregates, (NDS and MCS). The aggregate obtained from the resources were sieve to separate the aggregate into different sizes for later use. Washed sieve analysis was done to determine the percentage of dust and silt-clay material in order to check the need for filler material. Also, the physical and chemical properties for bitumen binders and mineral fillers were investigated.

The second series involved the mix design. A total of 224 specimens (Marshall and Superpave) were prepared. The sample preparation incorporated the mixing and compaction temperatures, and determining the optimum bitumen content. The Rice method was used to determine the maximum theoretical specific gravity, and water displacement method was used to determine the bulk specific gravity.

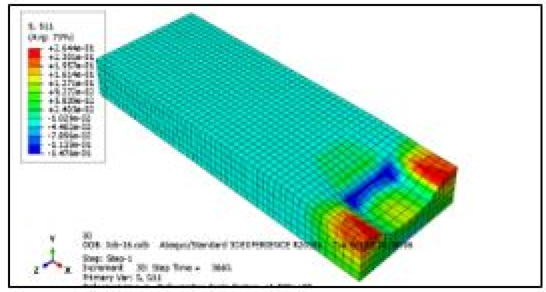
The tests were performed on the samples as following: SUPERPAVE Gyratory Compactor (SGC), Marshall Stability, Rutting Analyzer, Complex Modulus, Indirect Tensile Stiffness Modulus (ITSM), Moisture susceptibility (TSR), and Creep. These tests provided useful insight into the asphalt's deformation behaviour in-service. All tests accomplished in highway and pavement laboratory, École de Technologie Supérieure (ETS), University of Quebec.

2.3.4 Determining the Optimal Ratio of Natural Desert Sand

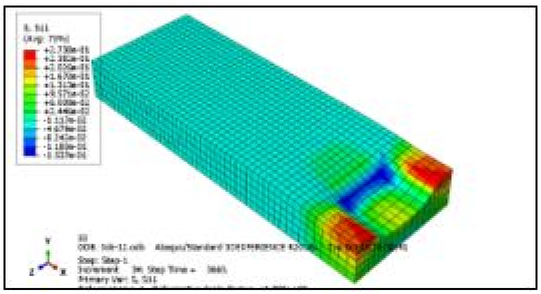
Rutting testing equipment and procedures were used to determine the amount of natural sand required. For all of these tests, both PG70-10 and B60/70 were used. The tests were carried out with 50% natural sand content, but this mix was found to be insufficient in terms of strength and stability. Following tests were carried out with 25% natural sand content and these showed that the amount of natural sand was not a problem in terms of the final mix quality. Nonetheless, the objectives of the project are to find a workable maximum amount of natural sand; therefore, more tests were done. Following tests used 30% and 35%. At 30%, the resulting pavement showed good characteristics but at 35% the tests showed too much deformation. As a result, a natural sand content of 33% by weight was selected for the samples in this study with regard to the total mass of the aggregate mixture. See Figure 2.1.



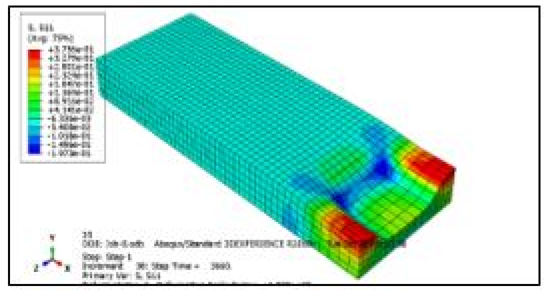
25% of NDS Content



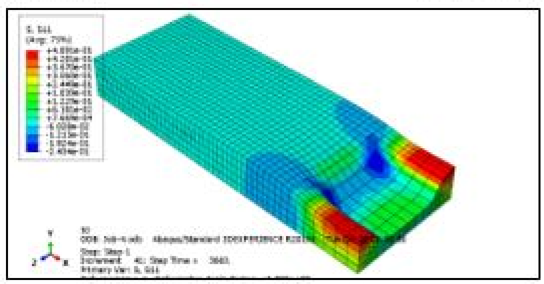
30% of NDS Content



33% of NDS Content



35% of NDS Content



50% of NDS Content

Figure 2. 1 Rut depth deformation after loading

2.3.5 Operational Framework

The following flowchart in Figure 2.2 shows the steps in the Superpave testing program. Mixtures results and analysis for the laboratory works were discussed in the Chapters 3, 4, and 5.

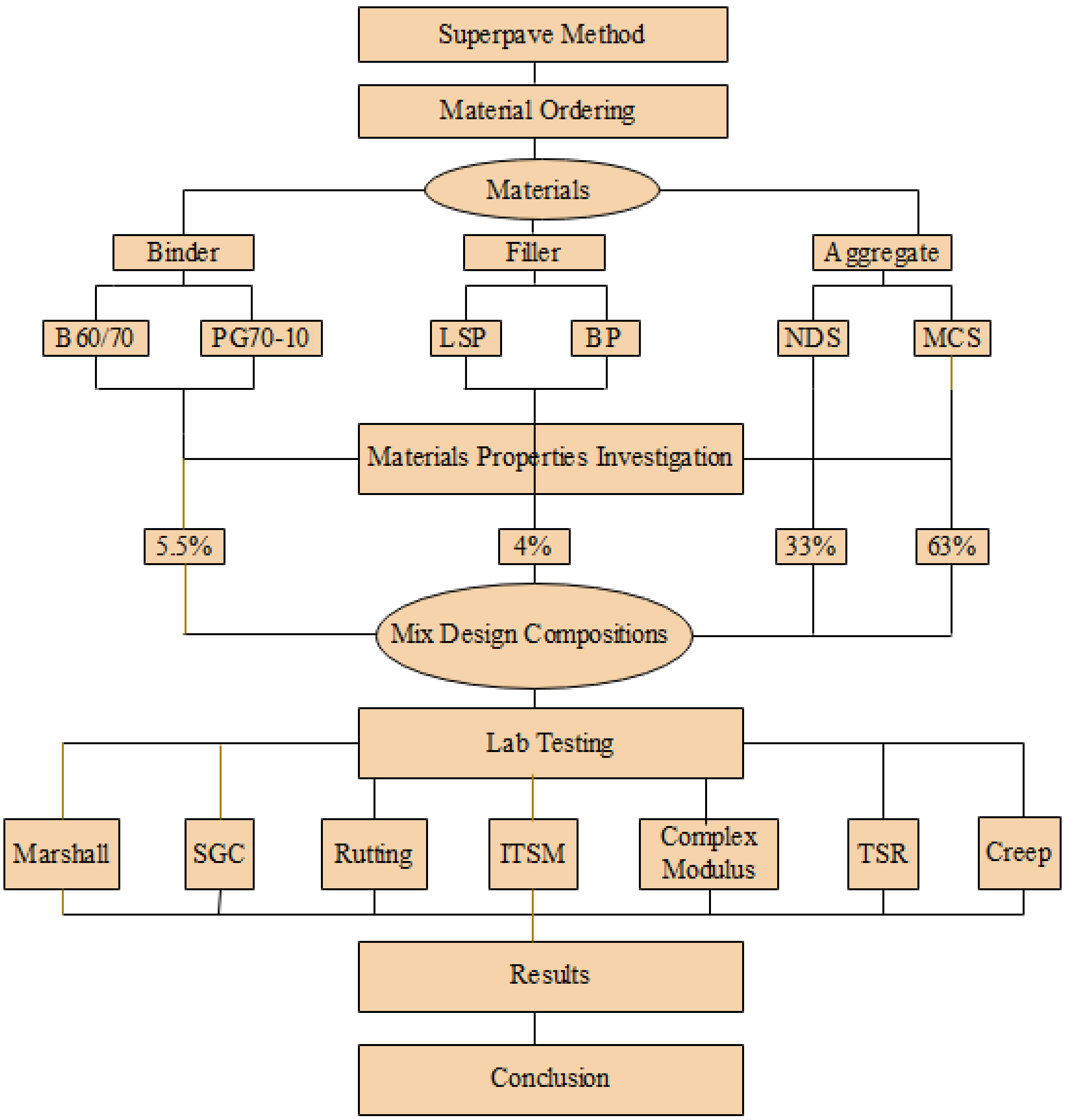


Figure 2. 2 Flow diagram for laboratory analysis process

.

2.4 Research Significance

ACP maintenance and construction must be cost effective. Another increasingly important requirement in ACP construction is to minimize environmental impacts, including the use of non-renewable resources and replacing them with more energy efficient and environment-friendly alternatives. As such, this thesis explores innovative practices in ACP construction to take advantage of locally available desert sand as a material in the construction of LVRs. This also minimizes delays in construction due to transport. Other contributions include using recycled materials, namely brick powder, both for environmental reasons but also because it has better performance characteristics than some other non-renewable resources.

2.5 Outline of Thesis

This Ph.D. thesis is a manuscript-based thesis separated into different sections plus an introduction. In this regard, Chapter 1 covers the literature review with an emphasis on the effects of adding natural sand to ACP. The problem statement, the objectives and the outline of the research are explained in Chapter 2. Chapter 3 explains the effect of using two different binders on HMA. It is also available and published in the Journal of Construction and Building Materials (Elsevier). Chapter 4 demonstrates the effects of natural sand on rutting deformation and complex modulus prediction formulas. It is under review in the Journal of Structural Engineering and Mechanics (SEM, Techno press). Chapter 5 demonstrates the mix design improvement technique to replace the filler in the mix. It is under review in the journal of Materials in Civil Engineering, American Society of Civil Engineers (ASCE). After that this research covers the discussion and conclusions. Appendix I and II are additional conference papers associated to the findings of this research.

2.6 Statement of originality

This project has four main contributions.

First, it investigates the effect of bitumen type on the rheological properties of asphalt mixtures. Different mixtures were made containing Performance Grade (PG70-10) and Penetration Grade (B60/70) bitumens, containing the same gradation of crushed sand and natural desert sand. The finding show that the PG70-10 mix is superior and a more effective mixture to combine with natural desert sand. HMA with PG70-10 has a high level of stability and decreased flow, which leads to fewer instances of rutting and an improvement in the overall durability of the mixture in hot and arid regions.

Second, the effects of natural desert sand (NDS) as a fine aggregate in hot mix asphalt (HMA). Marshall test results showed a lower air void percentage and more permanent rutting deformation. The results of the IAPST tests showed a decrease in V10 and V50 because the NDS particles were rounder and smoother than the manufactured crushed sand (MCS). 33% NDS in HMA was optimal based on the results that founded in rutting tests with binder bitumen PG70-10 which subjected to compaction and then to steady-state deformation.

Third, the effect of filler type on HMA mixture properties was tested. Here, the HMA was made with brick powder instead of limestone as filler material. The results showed that the mixtures using brick powder had superior mechanical characteristics to mixtures using limestone. The stiffness modulus tests were best for differentiating brick powder from limestone. Brick powder filler may also limit permanent deformation through creep tests at high temperatures; this allows for better resistance to rutting in HMA in extremely hot climates.

Fourth, different HMAs were tested that had different ratios of natural sand to crushed sand. The findings show the sensitivity of the ratio on both mixture stability and stiffness at different temperatures. The optimal percentage of natural sand is consistently at 33% of the total mass of the mixture, where the remaining mix is comprised of 63% crushed sand and 4% filler.

CHAPTER 3

EFFECTS OF ASPHALT BINDERS ON PAVEMENT MIXTURES USING AN OPTIMAL BALANCE OF DESERT SAND

Fathi S. Almadwi[(1)] and Gabriel J. Assaf[(2)]

Department of Civil Engineering and Construction, École de Technologie Supérieure (ÉTS), 1100 Rue Notre-Dame Ouest, Montréal, QC H3C, University of Québec, Canada.

Paper published in *Journal of construction and building materials*, June 2019

Abstract

Libya's hot and arid desert regions create two serious constraints for Asphalt Concrete (AC); the temperature of the road surface can be as high as 70°C and the humidity can be very low, resulting in premature AC deformation and cracking. This research demonstrates the effects of two different bitumen binders as variables against the control of a predetermined fine aggregate mix. The mixes are subject to several tests, including the Marshall, Superpave, Rutting Analyzer, and Complex modulus tests on AC. All samples were tested under simulated Libyan climatic conditions. These tests were conducted with an aggregate mix containing only 63% manufactured sand and 33% natural desert sand; this mix was used to test two asphalt binders, the widely-used B60/70 and the new PG70-10. As expected, all the tests performed on the B60/70 and PG70-10 mixes show that the PG70-10 mix is superior. Criteria for low-volume roads (LVRs) are met with the PG70-10 mix; this mix shows practical advantages for hot and dry regions where the use of locally available natural sand can result in substantial savings. For low-volume roads in remote areas, these findings are particularly important because economics do not justify the use of costlier higher-grade materials.

Keywords: Hot regions, Marshall, Superpave, Rutting, Complex modulus, Desert, LVRs, AC.

3.1 Introduction

In arid and hot regions, such as in North African or Middle Eastern countries, roads often deteriorate more quickly than in temperate climates. These road surfaces suffer from poor mix designs resulting in substantial deformation (e.g., rutting, shoving), especially in desert regions. The binder now used in AC for these roads is Bitumen 60/70 (B60/70). This binder is often made in refineries in Libya, but it does not perform well in very hot conditions, resulting in substantial shear failure. Daytime air temperatures in the deserts can be as high as 49-54°C, leading to road surface temperatures of over 70°C (Salem et al., 2014). The Marshall test is useful for assessing material qualities under standard conditions, but it does not measure the fundamental rheological properties of the asphalt mix (Almadwi & Assaf, 2017).

Southern Libya, like any area with arid sand deserts, has practical reasons for using local dune sands in asphalt mixes; nevertheless, too much desert sand aggregate in an AC results in an AC that is very likely to show stresses resulting in permanent deformation under use (Almadwi and Assaf, 2018). Over the course of time, desert sand particles become rounded through years of grinding against each other. Therefore, these sand particles are more rounded than comparable-sized newly-made aggregates. For practical purposes, the functional properties of the resulting AC are also markedly different. The manufacturing of AC mixtures with a high ratio of natural sand have attracted attention as a means of cost cutting and resource saving by efficiently using the existing natural options (Almadwi and Assaf, 2018).

3.2 Background

In hot regions, pavement failure can take many forms, such as rutting and fatigue cracking. Rutting is defined as a permanent deformation that appears as longitudinal depressions in the wheel paths of roads; typically, such deformations are long. Rutting greatly reduces the service life of a road because it makes driving less comfortable and can be a hazard to vehicle navigation. Causes of rutting fall into the following important categories: material selection, asphalt mix design, construction, axle-load, and climatic factors (Abdulaziz A. Bubshait, 2001). Some asphalt pavements remain sound for many years and later show signs of rutting. In such cases, rutting is often caused by an increased volume of truck traffic. To bring the pavement up to a level where it can meet these increased demands, asphalt material selection, pavement mix design, and construction techniques should all be well addressed (Breakah et al., 2011).

Asphalt pavement mixes are complex structures made up of bitumen, aggregates and air voids. In this context, the properties of binders and fine aggregates are crucial; the fine aggregate is defined as the aggregate passing through the US sieve No. 4 (4.75 mm) and retained on sieve No. 200. These three components (binders, aggregates, and air voids) work together to create the finished pavement's load carrying capacity and are key factors with regard to the pavement's expected working lifespan (Al-Hassan, 1993). "The breaking of the adhesive bond between the aggregate surface and the bitumen" is called stripping. Stripping is dependent on many factors, including the use of anti-strip additives, the type and use of the mix, asphalt characteristics, aggregate characteristics, traffic, environment, and construction practice. However, the common factor to all stripping is the presence of moisture. The asphalt binder can affect the moisture susceptibility of the mixture as well, since it impacts both the adhesion between the aggregate particles and the cohesion of the mastic (Asif et al., 2018). The structure is created when these factors interlock, i.e., how the binder interacts with the individual aggregate fragments to cause them to interlock and, with the air voids, creates a mobile structure with the target load carrying capacity. This microstructure, in turn, determines how the pavement resists permanent deformation. To maintain sufficient air voids in the asphalt mix, the film thickness of the bitumen on each aggregate particle must be considered. Without appropriate film thickness, there will be a collapse of the overall asphalt bearing capacity and a collapse of the air-void content; this leads to less overall elasticity in the road matrix and, eventually, severe rutting. Broadly, the more the desert sand used, the lower the air void content level and the shorter the predicted pavement lifespan (Al-Hassan, 1993).

The effects of natural sand on asphalt pavement have been examined in (Niazi, Y., & Mohammadi, 2003). In this study, the following aggregate blends were used with B60/70:

manufactured sand, natural river sand, and a mix of the two. Their findings showed that these mixes did not have the sensitivity needed for the best results. In (Ahmed, 2016) the effects of various natural sand mixes were tested with regard to fatigue effects on hot mix asphalt (HMA); the study used two fine aggregate types, manufactured sand and natural desert sand, where the desert sand is used in the following percentages: 0%, 25%, 75% and 100%, by weight of the total aggregate mass. The study used only one binder (B40/50). These tests found that the mixture with 0% and 25% natural desert sand were the best.

For the present project, the consideration of how this structure is affected by high temperatures is critical because the viscosity of the binders is much lower at extreme temperatures, causing a softening of the binder and contributing to a more permanent deformation, such as rutting. In turn, the permanent deformation depends on the methods used for testing and preparing specimens, including loading times, testing temperatures, and stress levels. The advances seen in the field of pavement design in the recent past are very significant (Epifanio & Gan, 2009).

Compaction plays the greatest role in the performance of the asphalt pavement and must be considered from the perspective of construction techniques and material selection. Compaction involves a number of factors, all affecting the density of the finished pavement. Such factors include: mixing temperatures, compaction patterns, rolling conditions, rolling equipment, driver experience, and local climatic conditions (Al-Hassan, 1993). In order to attain suitable compaction and mixing, every component in the AC must be carefully determined because they have an effect on the AC’s characteristics, including its response to moisture, permanent deformation and fatigue cracking. In testing, other factors that must be included so as to accurately model real-world conditions are axle-load distribution and climate (Epifanio & Gan, 2009).

Exact bitumen content is essential to creating the targeted air void content in the final mixture, because it affects everything from AC flexibility, stability, durability, and the degree to which it will be susceptible to moisture damage. Imprecise bitumen levels result in a mix that is too dry, leading to early cracking and raveling, or a mix that is too rich, leading to deformations such as rutting. At the same time, when there is excessive bitumen, the finished road will be subject to bitumen bleeding and more deformation, particularly under hot conditions, with high traffic loads (Roberts, F. L., Kandhal, P. S., Brown, E. R., Lee, D. Y., & Kennedy, 1996).

Performance Grading (PG) can be defined as a means of specifying a binder for asphalt pavement with regard to its performance at certain temperatures, i.e., temperature affects the stability, flexibility, durability, fatigue resistance, and resistance damage from moisture. A material can only be predicted to meet the requirements of a given project within a limited scope; PG is used to specify this temperature range (Hay, Richard , E., & Peter, 1986).

Aggregate quality and consistency are critical to pavement performance, and natural or crushed sand normally serves as the fine aggregate in asphalt mixes. These fine aggregates have an important influence on the performance of asphalt mixes, especially with regard to rutting, although there is disagreement on the optimal balance of materials. In high temperature regions asphalt concrete mixes can incorporate natural desert sand aggregates or partially crushed aggregate. Aggregate weight in asphalt mixes is 90-95% and aggregate volume is 75-85%. The performance of an asphalt pavement, including its load-bearing capacity, as well as how easy the mix is to work with during construction, depends directly on the physical characteristics of the aggregates used in the HMA (Golalipour, Jamshidi, Niazi, Afsharikia, & Khadem, 2012). As such, in designing a quality pavement that will last many years, is important to have an effective means of selecting the most appropriate aggregate (Asphalt Institute, 1998). Therefore, to design a pavement that will resist permanent deformation and have a long service life, the planners must consider the structural matrix formed by the aggregate. This structure is essential to receiving and dissipating energy that comes from axle loads on the pavement surface. It is critical that the matrix can absorb and dissipate this energy without undergoing structural changes itself. In analyzing an aggregate for its characteristics, the coefficient of friction of the pieces of fine aggregate need to be considered; this, as well as their form and the gradation of the whole will determine the qualities of the matrix structure (Chowdhury, A., Button, J. W., & Grau, 2001).

For practical planning purposes, the balance between natural sands and manufactured sands determines the angularity and, therefore, the structural matrix of the finished pavement. As such, there are regulatory guidelines that help planners determine suitable mixes. For instance, the Federal Highway Administration of America (FHWA) specifies natural sand to make up about 20% of the fine aggregate, whereas other US jurisdictions require about a 43-45% fine aggregate angularity (FAA) for bituminous mixes on high traffic roadways. This can be compared with Chinese road regulations specifying that fine aggregates in AC should consist of both quality natural or crushed sands (Wong, Gordon, 2016). Furthermore, Superpave asphalt mixes specify FAA as the principle criterion for gauging the quality of fine aggregates. Depending on testing conditions, there is some controversy about whether or not the FAA values are the primary factor affecting asphalt mix performance. Nonetheless, in remote areas, quality aggregate can create much higher costs for construction due to transportation costs. This is especially for repairs and small projects. Prior research has found that there is no conclusive connection between the FAA and the performance characteristics of HMA. These findings also reported that it was not only the shape of the fine aggregate but also factors such as specific gravity, particle strength and surface texture that contributed to the performance qualities of the aggregate in the HMA (Breakah et al., 2011).

The voids in the loosely packed fine mineral aggregate correspond to the numerical value of the FAA. More angular particles have a higher FAA value, and as such, the air voids are larger. The practical function of the FAA is predicated on the idea that fine aggregate with higher and lower FAA values pertain, respectively, to fine aggregate with higher or lower friction. Particle shape and texture directly affect the fine aggregate when it forms a matrix, where rutting resistance in a finished asphalt road corresponds to higher friction in the aggregate. Levels of FAA in the Superpave system are grouped in three levels, angularity below 40, angularity between 40 and 45, and angularity above 45. The Superpave protocol specifies higher values for layers nearer the surface as well as for higher levels of traffic. At the same time, fine aggregate angularity may not be sufficient to determine how fine aggregate affects the performance of the HMA. Gradation, affinity for asphalt, absorption and other factors also affect the performance of the HMA (Lee, C. J., White, T. D., & West, 1999).

Asphalt roads rut prematurely rather than crack in hot and arid desert regions and their functional life is a lot shorter than in temperate areas. Superpave techniques determine the highest temperature of the performance grade and they are more suitable for higher axle-loads than Marshall methods, but the Superpave techniques need to be adapted to reflect the higher temperatures prevalent in the field.

This paper attempts to demonstrate the effects of two different bitumen binders as variables against the control of a predetermined fine aggregate mix. The mixes were subject to several tests, including the Marshall, Superpave Compactor, Rutting Analyzer, and Complex modulus tests. The tests were conducted with the same equipment and the results are used to identify which mix is more suitable for the conditions in question. The mixes only included fine aggregate from desert and manufactured sands. The results are analyzed and discussed to determine the specific effects of the binders on the performance of the asphalt.

3.3 MATERIALS AND TESTING PROCEDURES

3.3.1 Materials

Two types of binders were used in this study, PG 70-10 and B60/70 asphalt binders. The physical characteristics of the two bitumen binders are shown in Table 1. Figure 1 illustrates the Brookfield viscosity with regard to temperature equivalents and how they relate to the binders used here. Asphalt layers in pavement construction are usually made up of at least two different layers with each layer having a different thickness: wearing course, binder course, and base course. This study used samples from the sand dune area in the southwestern desert of Libya, around the city of Sabha. These samples were mixed with manufactured sand, with particles from 0-5 mm. This study used an asphalt mixture with a nominal maximum fine aggregate size of 5mm; this is called semi-grouted asphalt (ESG-5), and it is normally used as a surface layer according to Ministère des Transports de Quebec guidelines. The volumetric properties for the natural sands, the manufactured sands and the limestone filler are shown in Table 2. The asphalt surface layer is the layer that must resist deformation. The surface layers play a major role in top to bottom rutting deformation resistance. The aggregate gradation specification for HMA is shown in Table 3. Figure 2 contains aggregate gradation specification for HMA.

Table 3. 1 Asphalt binder properties

Binder Properties	Limits		Standard Specification	Samples Result	
	PG70-10	B60/70		PG70-10	B60/70
Specific Gravity	1.029	1.03	ASTM-D70 (T228)	1.0285	1.03003
Viscosity, Rotational, 135°C, Pa·s	≤3	-	ASTM-D4402 M-15 (T316)	0.394	-
Dynamic Shear, 70°C, G*/sinδ, kPa	≥1.00	-	ASTM-D7175 (T315)	2.20	-
Flash point, °C	≥230	≥235	ASTM-D92 (T48)	284	302
Mass Loss, %	≤1.00	≤0.2	ASTM-D113 (T240)	0.03	0.07
Creep Stiffness, 0°C, S, MPa	≤300	-	ASTM-D6 D6648 (T313)	102	-
Solubility, Min. %	≥99	-	ASTM-D4 D2042 (T44)	99.96	-
Penetration at 25°C, dmm	-	60-70	ASTM-D5 D4402 (T316)	-	64.7
Ductility at 25°C, cm	-	≥104	ASTM-D113 D70 (T51)	-	143
Softening point, °C	-	48-56	ASTM-D36 (T53)	-	51.7

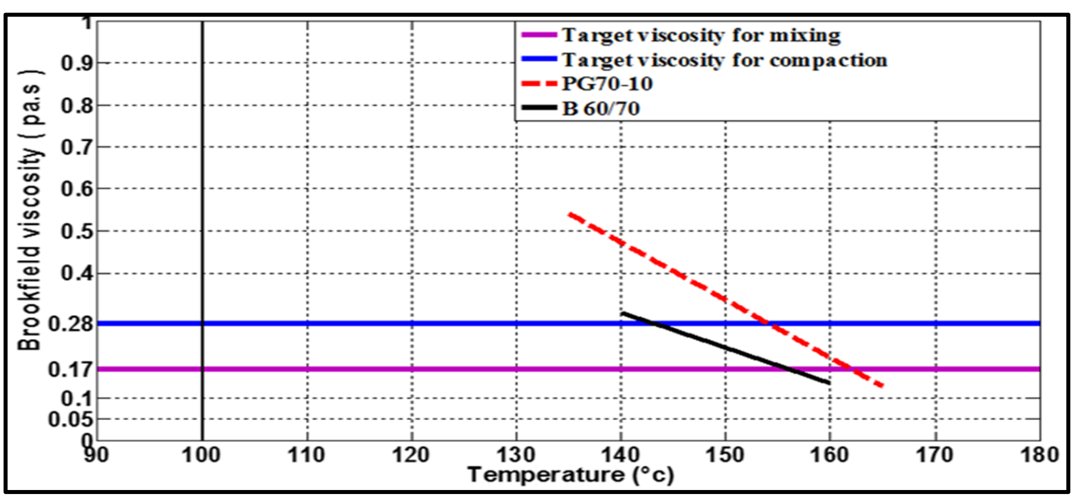


Figure 3. 1 Brookfield temperature vs viscosity curve for binders

Table 3. 2 Aggregate Properties

Aggregate	Apparent Specific Gravity	Bulk Specific Gravity	(%) Absorption
Natural Sand	2.63	2.42	0.33
Manufactured Sand	2.639	2.44	0.58
Limestone Filler	2.70	-	1

Table 3. 3 Asphalt mix composition and volumetric properties

Agg size (mm)	Requirements	ESG5, % passing (of 33% natural sand & 63% manufactured sand)				
		Minimum Limit	Maximum Limit	Mix Composition Passing (%)	Manufactured Sand	Natural Sand
14.00	100	100	100	100	100	100
10.00	95-100	95	100	100.0	100	100
5.00	85-100	85	100	95.6	96.5	98
2.50	50-70	50	70	65.4	51.6	68.3
1.25	-	24	34	29.6	25.2	31.8
0.63	-	16	24	17.4	16.5	23.7
0.32	-	14	22	14.1	14.9	21.2
0.16	-	8	12	9.8	8.6	11.2
0.080	4-8	4	8	5.4	4.8	5
Volumetric properties of the mixture						
G_{mm}	2.514					
G_{sb}	2.435					
V_{be} (%)	12.2					
B_c (%)	5.5					

Were

G_{mm} Theoretical gravity max.

G_{sb} Bulk specific gravity.

$V_{be\,(\%)}$ Effective binder volume.

$B_{c\,(\%)}$ Bitumen content.

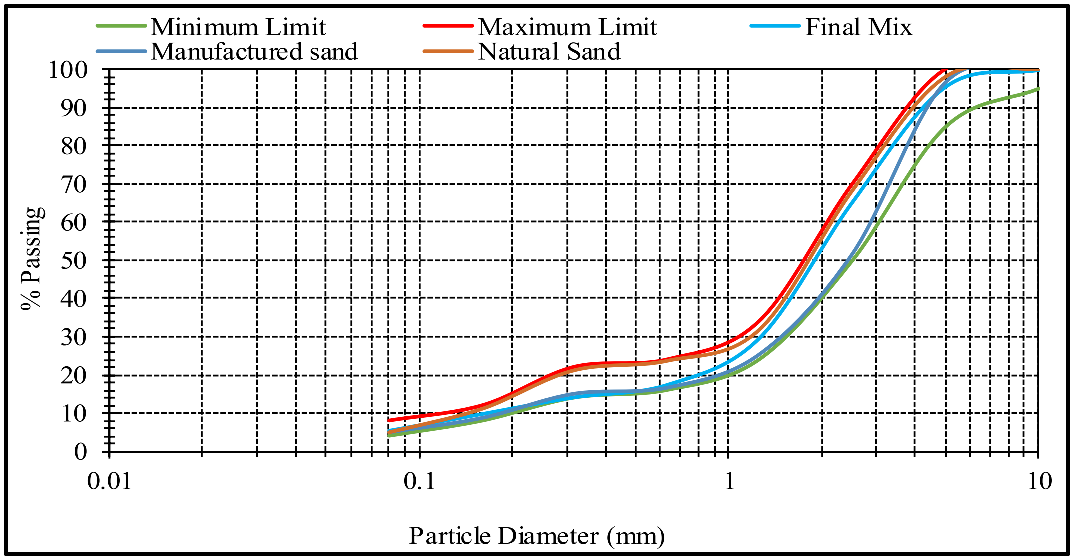


Figure 3. 2 Aggregate gradation specifications for hot mix design

3.3.2 Test procedure

The tests were conducted on two AC mixtures; the first mixture used PG70-10, and the second one used B60/70. Both included natural desert sand by weight of the mix, which was kept constant at 33%. Using the Marshall method the optimal bitumen content (OBC) was determined. The tests were carried out with 4.5% bitumen content, but these were found to be insufficient to bind the mix. Following tests were carried out with 5%, 5.5%, and 6%. The test with 6% bitumen resulted in too much bitumen bleeding. As a result, bitumen content of 5.5% by weight was selected for the samples in the following tests. About 4% of limestone filler was also added and was constant in both mixtures.

3.3.2.1 Marshall Test

The Marshall test followed (ASTM D-6927) in order to determine the OBC for compacted asphalt samples with 4% air voids. Four specimens for each of two mixtures were prepared by putting the mix in a preheated Marshall mold and then using a compaction hammer to compact it with 50 blows on each of the two sides of the sample. The following volumetric properties of the samples represent the average of the four samples as determined by the tests: the percentage of voids filled with asphalt (VFA); the percentage of air voids (Va); the percentage of voids in the mineral aggregate (VMA); and the stability and plastic flow.

3.3.2.2 SUPERPAVE Gyratory Compaction (SGC) Test

At each stage of the compaction procedure, the tests produced compacted specimens that provided volumetric properties, air void (Va), voids in mineral aggregate (VMA), and voids filled with asphalt (VFA). The samples were evaluated at the following intervals: 10, 80, 100 and 200 gyrations. ESG-5 category standards for hot mix paving materials were taken from the (Ministère-des-Transports-de-Quebec, 2016). These standards specify targets of Va ≥ 11% for N_{ini}, Va = 4 – 7 % for N_{des}, and Va ≥ 2 % for N_{max}.

3.3.2.3 Rutting analyzer Test

The test samples were subjected to a loaded wheel test, shown in Figure 3, to simulate axel-loads over time. The machine temperature was adjusted because the binders had different softening points; likewise, wheel weight and tire pressures were adjusted to simulate various axle loads. With a standardized room temperature of ~20°C, the initial test specified 1000 cycles with a tire pressure of 600kPa; following tests were performed at 1000, 3000, 10,000 and 30,000 cumulative cycles at 65°C. According to the standards specified in (Ministère-des-Transports-de-Quebec, 2016), at 65°C the sample can lose up to 10% of its original height (in mm) at 1000 heated cycles and 15% at 3000 heated cycles.

Figure 3. 3 Specimen preparation for the rutting tests

3.3.2.4 Complex modulus test

Several rigorous and targeted modulus tests were undertaken on cylindrical samples that had a height of 150 millimeters and a diameter of 75 millimeters; these samples were fixed to a hydraulic press by means of two aluminum caps at each end and attached with an epoxy adhesive. To measure the axial displacement, three axial extensometers were situated at 120° in relation to the sample. Three measurements were used to record the axial strain and were also used to monitor the applied loading. Figure 4 shows the specimen and the strain measurement schematics taken with the instruments discussed here (Ministère-des-Transports-de-Quebec, 2016).

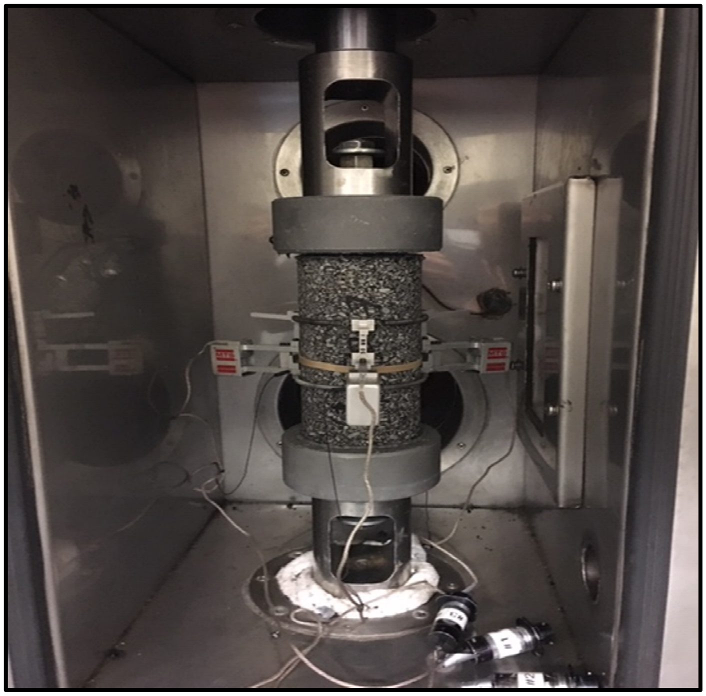


Figure 3. 4 Specimen preparation and strain measurement sectioning

To find a wider range of viscoelastic characteristics of the materials, the samples were measured at eight temperatures ranging from −25°C to 50°C: (−25, −15, −5, 5, 15, 25, 35, and 50) and loaded at seven frequencies (0.01Hz, 0.03Hz, 0.1Hz, 0.3Hz, 1Hz, 3Hz, and 10 Hz). These measurements were carried out under strain control settings. Axial strain amplitude through cyclic loading came to 50 μ-strain. Frequencies and number of cycles for the tests are shown in Table 4.

Table 3. 4 Frequencies and number of cycles for temperatures tested

Frequencies (Hz)	0.01	0.03	0.1	0.3	1	3	10
Number of cycles	16	16	20	20	30	50	100

Between each temperature change and in preparation for a new series of conditioning tests, there was conditioning period of approximately four hours to bring the sample to the correct test temperature. This conditioning period was used to ensure a homogeneous temperature throughout the sample. There was no load on the sample during this period. Then the cyclic loading began. Between each successive cyclic loading period, there was a rest period; these rest periods punctuated every change in frequency. The total rest period for each testing temperature was approximately 1 hour and 20 minutes. It took a total of approximately 72 hours for a complete complex modulus test.

Sinusoidal axial stress **(σ_{ax})** was evaluated with a load cell, and the three extensometers were averaged and then applied in terms of a sinusoidal axial strain (ε_{ax}). These quantities are shown in the equations below (Hoang et al., 2015):

$$\varepsilon_{ax}\,(t) = \varepsilon_{Aax}\sin(\omega t) \tag{3.1}$$

$$\sigma_{ax}(t) = \sigma_{Aax}\, sin\,(\omega t + \varphi_E) \tag{3.2}$$

Where

φ_E Is the phase angle as taken from values for axial stress and axial strain;

ε_{Aax} and σ_{Aax} are amplitudes pertaining to axial strain and stress amplitudes.

In this test, the complex modulus (E^*) defines the relation between strain and stress during continuous sinusoidal loading. The complex modulus is the norm of the modulus corresponding to the stiffness of the material, and this is characterized in terms of the dynamic modulus $|E^*|$ (Badeli, Carter, & Doré, 2018).

$$|E^*| = \frac{\sigma_0}{\varepsilon_0} \tag{3.3}$$

Where

σ_0 Max. stress amplitude;

ε_0 Peak recoverable strain amplitude.

(φ) represents the phase angle, which is a phase lag of the strain creating the stress from the solicitation of the sinusoidal cycle. A phase angle of 0° would represent a completely elastic material, whereas a phase angle of 90° would represent a completely viscous material. The sinusoidal stress of pulsation (ω), in terms of a ratio of the amplitude, is written as $\sigma = \sigma_0 \sin(\omega t)$, just as the sinusoidal strain's amplitude is written as $\varepsilon_t = \varepsilon_0 \sin(\omega t - \varphi)$, resulting in a steady state (Di Benedetto, H., Olard, F., Sauzéat, C., & Delaporte, 2011).

$$E^* = \frac{\sigma}{\varepsilon} = \frac{\sigma_0 e^{i\omega t}}{\varepsilon_0 e^{i(\omega t - \varphi)}} \tag{3.4}$$

3.3.2.4.1 Linear viscoelastic (LVE) analysis

To analyze the HMA's structural behaviour with regard to its evolution over time in laboratory conditions, the complex modulus test (|E*|) was used to investigate the thermomechanical properties of the specimen's small strain domain amplitude. To ensure that the measurement of the sample remained within the LVE domain, the target was fixed at 50 μ-strain. The complex modulus (|E*|) can be seen in extremely small strain amplitudes and it corresponds to the mixtures' LVE behaviour. Figure 5 illustrates the various behaviors of bituminous mixes with regard to the strain amplitude and the loading cycles. The illustrated boundaries are for the different behaviors, shown here in terms of orders of magnitude. These may vary sizably according to the temperature, material, and the loading path direction (shear, compression, etc.).

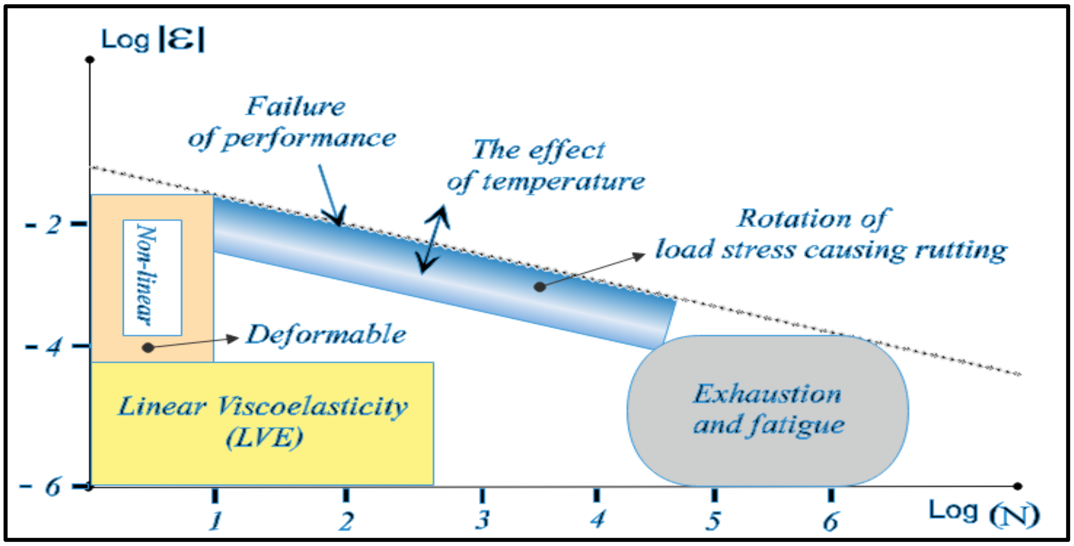


Figure 3. 5 Various ranges of behaviour seen in the asphalt mix

3.3.2.4.2 Linear Viscoelastic 2S2P1D Model

2S2P1D modeling (i.e. two springs, two parabolic elements, and one dashpot) is shown below in Figure 6. This is a common model to help create a linear understanding of how bituminous materials such as mastics, mixes, and binders behave in viscoelastic, unidimensional, and tridimensional ways (Mangiafico et al., 2014).

Equation 5 was used to analyze the complex modulus results with the 2S2P1D rheological model.

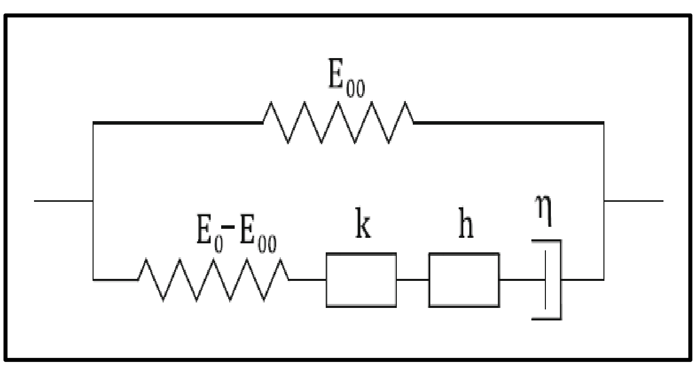


Figure 3. 6 2S2P1D model

The following equations give the complex modulus, at a determined temperature which was discussed by (Olard & Di Benedetto, 2003):

$$E^{*}_{2S2PID} = E_0 + \frac{E_{00} - E_0}{1 + \delta(i\omega\tau)^{-k} + (i\omega\tau)^{-h} + (i\omega\beta\tau)^{-1}} \tag{3.5}$$

Where

E_0 the static modulus when $\omega \to 0$

E_{00} the glassy modulus when $\omega \to \infty$

ω is the pulsation, $\omega = 2\pi f$ (f is the frequency)

k, h constant such as $0 < k < h < 1$: δ constant

β parameter linked with η,

the Newtonian viscosity of the dashpot,

$$\eta = (E_0 - E_{00})\,\beta\tau_E \tag{3.6}$$

τ_E characteristic time values; these are temperature-dependent parameters and show a similar evolution, represented by the following equations (Olard & Di Benedetto, 2003):

$$\tau_E\,(T) = a_T\,(T)\,.\,\tau_{OE} \tag{3.7}$$

Where

$\boldsymbol{a_T}$ is the shift factor at temperature T.

At reference temperature

$$T_{ref}, \tau_E = \tau_{0E} \tag{3.8}$$

The constantsE_{00}, E_0, δ, k, h, β, and τ_{0E} characterize 3D LVE properties of the samples at the test temperatures. WLF law (equation 9) approximates the evolutions of $\boldsymbol{\tau_E}$ such that $\boldsymbol{\tau_E}$ at a given temperature T_{ref} is shown as$\boldsymbol{\tau_{0E}}$. Once the temperature effect is determined, the constant criteria become nine; this accounts for the two WLF constants (which are indicated as C_1 and C_2, based on the reference temperature) (Olard & Di Benedetto, 2003).

$$log\ (a_T) = -\frac{C_1(T - T_{ref})}{C_2 + T - T_{ref}} \tag{3.9}$$

3.4 Result and discussion

3.4.1 Marshall test results

All the Marshall tests were performed with 33% natural dune sand and 63% crushed aggregate (manufactured sand). The results indicate that PG70-10 is the preferred binder for the described conditions. This is supported by several observations. As seen in Table 5, the Marshall test samples using PG70-10 met the required targets. The tests with B60/70 were satisfactory, except for the Flow test. Furthermore, the percentage of Air Voids (Va) in the natural sand and PG70-10 mixture was within 4% (i.e. the acceptable limit), but the Va% of natural sand and B60/70 exceeded this recommendation. In the VMA, VFA, and Stability tests, both mixtures were within allowable limits. The Marshall Flow PG70-10 mix was within the acceptable range of 2-4.5 mm, whereas the B60/70 Marshall Flow mix passed this limit determined by the MTQ standard (Ministère-des-Transports-de-Quebec, 2016).

According to these results, PG70-10 is an effective mixture according to the Marshall Test standards, and surpasses the B60/70 mixture on a qualitative basis. B60/70 violated several of the specifications, which suggests that it is an inferior mixture in comparison to PG70-10.

Table 3. 5 Effect of binder-type on stability values for marshall asphalt mixes

Mix ID	VMA (%)	VFA (%)	Va (%)	Flow (mm)	Stability (N)
PG 70-10	15.8	77.9	3.15	3.75	22733
B 60/70	16.11	62.94	6.15	5.4	10149
MTQ Specification Limits	Min 15%	Max 85%	Between 3-5	Between 2-4.5	Min 9000

3.4.2 SGC Test Results

The SGC tests were performed on two samples with PG70-10 as a binder, and on another two samples with B60/70. Each of these two sample pairs were then averaged and can be seen in Figures 7 and 8. This test was conducted with targets of Va ≥ 11% for N_ini, Va = 4 – 7 % for N_des, and Va ≥ 2 % for N_max.

The Gyratory tests are shown below, and they illustrated that samples with the PG70-10 mix had a lower Va% than the B60/70 mix. This holds true for all phases of the gyration test. More critically, the PG70-10 mix had an air void content within the 4-7% target range for N-des at 80 and 100 gyrations. Both samples with Bitumen 60/70 exceeded the test criteria at 80 gyrations. The VMA test also exceeded the criteria set for the tests with the B 60/70 mixture. All other tests were within acceptable limits (Ministère-des-Transports-de-Quebec, 2016).

These test results also demonstrate the superior practicality of the PG70-10 mixture. The outcomes of the tests with PG70-10 were very positive, with the percentages of voids closely conforming to the specified targets. B60/70 was not as successful, with several factors exceeding the outlined criteria.

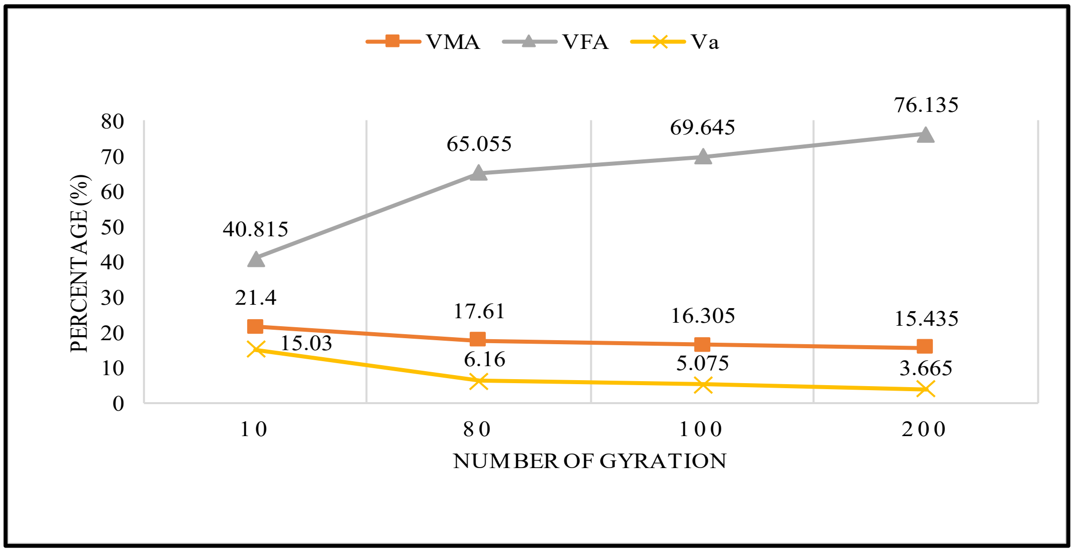


Figure 3. 7 SGC curves using PG 70-10

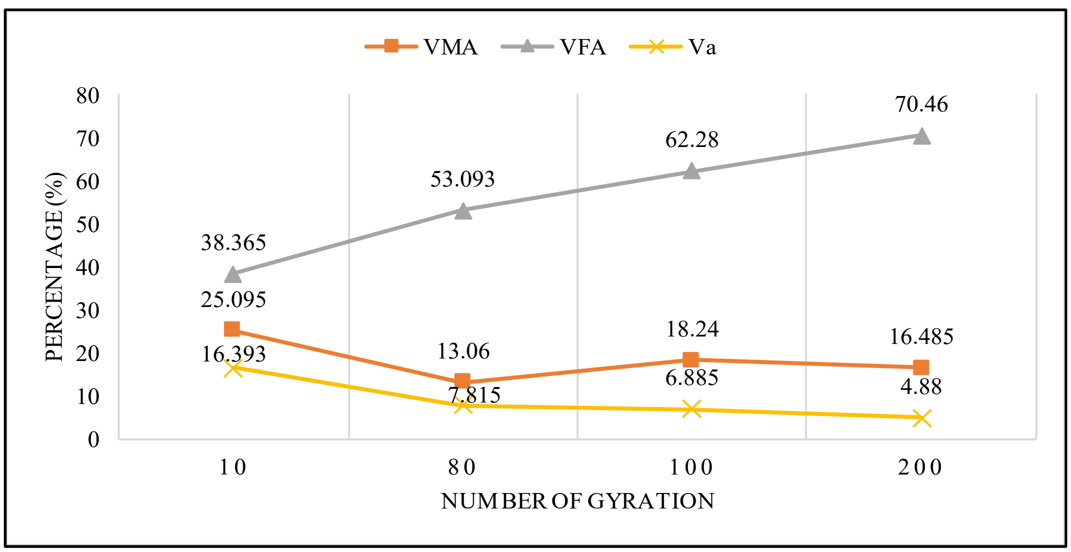


Figure 3. 8 SGC curves using B60/70

3.4.3 Rutting Analyzer Test Results

As illustrated in Table 6, samples using PG70-10 did not show as much rutting as samples using B 60/70; this result held for all rutting test stages (1000 – 30,000 cycles cumulative). The

PG70-10 samples, on average, lost 3.16 mm from their original height at 3000 cycles; the B60/70 samples, on average, lost 4.59 mm of their original height after 3000 cycles.

The most critical result of the rutting analyzer test, as seen in Table 5, is that the PG70-10 mix, from 1000 cycles to 10,000 cycles, has a slope that is clearly semi-straight and changes significantly after 10,000 cycles. Nonetheless, from 10,000 to 30,000 cycles, the results from the PG70-10 mix have a much lower slope than B60/70 mixes. The slope of the B60/70 mix from 1000 to 3000 cycles is clearly straight and increases significantly after 3000 cycles to the end. This has critical implications for an in-use road surface. Where the B60/70 mix undergoes considerable ongoing compaction, the PG70-10 mix shows much more general stability over time (Ministère-des-Transports-de-Quebec, 2016).

As shown in Figures 9 and 10, in general, results from the rutting analyzer test are expressed in terms of post-compaction consolidation around 1000 wheel-passes, stripping inflection point, creep slope, deformation values, and stripping slope. In a rutting analyzer test, the post-compaction consolidation stage shows the area on the sample where the initial wheel passes change the density of the asphalt mixture from the repetition of the applied load. The critical point of this consolidation stage (post-compaction) is where the rut depth curve versus the plot for the wheel passes disappears, and relatively straight region of the creep phase starts. Rutting is shown in the creep slope, i.e. the rate of permanent deformation in the asphalt. This represents the inverse rate of deformation, and it is the linear portion of the rutting deformation curve; this is the post-compaction consolidation point found prior to the start of the stripping. The stripping slope of the asphalt sample indicates moisture damage. The point where stripping and creep slopes intersect is the Stripping Inflection Point (SIP), pertaining to the number of wheel passes. Moisture damage is indicated by SIP, after which moisture damage begins dominating the asphalt mix performance.

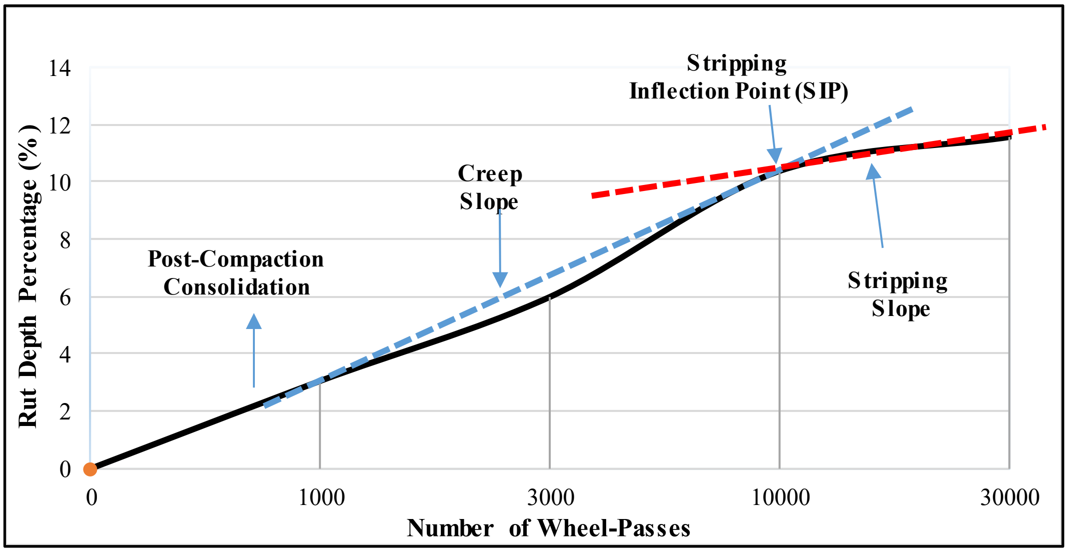


Figure 3. 9 Graphic of typical rutting test result of PG70-10 mixture

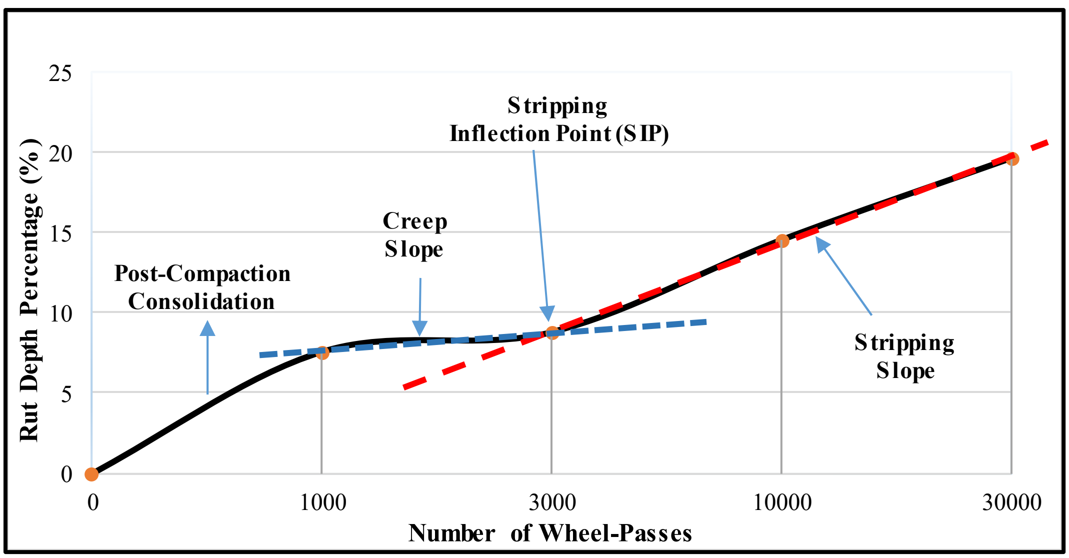


Figure 3. 10 Graphic of typical rutting test result of B60/70 mixture

Table 3. 6 Summary of rutting analyzer test results

Mix ID	Rutting depth at certain number of wheel passes (mm)				Creep slope	Post compaction	Average Va%
	1000	3000	10000	30000			
PG 70-10	1.60	3.16	5.46	6.11	12,980	1000	6.18
B60/70	3.97	4.59	7.64	10.33	2800	700	6.72

Table 6 shows the rutting depths at different numbers of wheel-passes along with the creep slopes, approximate post-compaction points, and the average Va% contents. The superior performance can be observed for PG70-10 mix, with the creep slope of 12,980 and the maximum rut depth of 6.11 mm after the limiting 30,000 wheel-passes, which confirms that the PG70-10 mix is superior to the others in terms of rutting as well as stripping performances. Similarly, rutting performances of B60/70 mix can be identified from Table 6 and Figure 10. As with the previous tests, B60/70 did not perform as well. The stripping point occurs at a much lower number of wheel passes, and stripping deteriorates more quickly. The post-compaction consolidation phase also begins earlier in the B60/70 tests.

3.4.4 Complex Modulus Test Results

3.4.4.1 Complex Modulus E*

Two mixtures were used for the Complex Modulus test: the aggregates were used as a constant and the variables were the bitumen binders B/6070 and PG70-10. The results from the analysis of the two samples help estimate the effect of the bitumen binders on the LVE properties of the samples. Variations in temperature, from the targeted to the recorded temperature, were corrected with the 2S2P1D model, providing satisfactory results at the planned temperatures. Two samples of each mix were tested, and the values were averaged. The Time-Temperature Superposition Principle (TTSP) is validated by the unique curve of a given specimen for all the materials by way of the LVE (Ministère-des-Transports-de-Quebec, 2016).

The Cole-Cole plane evaluates the results of the complex modulus mostly at intermediate and low temperatures while the Black curve explains the quality of the results mostly at high temperatures.

The results for the phase angle and the norm of the complex modulus ($|E^*|$ and φ_E), are given in Figures 11, 12 and 13. The 2S2P1D model curves were calculated with an average of the fitted curves for the samples of the mixes.

The diagrams of the Cole-Cole curves and of the Black space for the tested samples are represented in Figures 11 and 12. The imaginary plot of complex modulus (E^*) is shown in the Cole-Cole diagram as a function of its plot, whereas the norm of E^* in Black's space, as a function of the phase angle, is plotted by the curve on the graph.

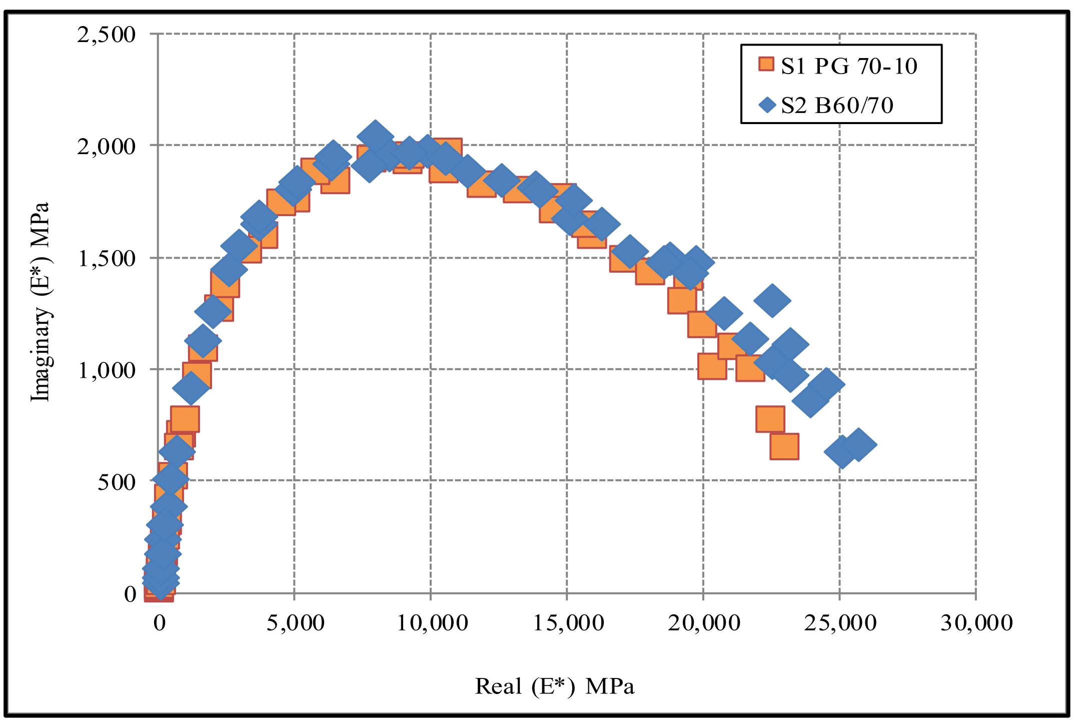


Figure 3. 11 Complex modulus results in Cole-Cole diagram

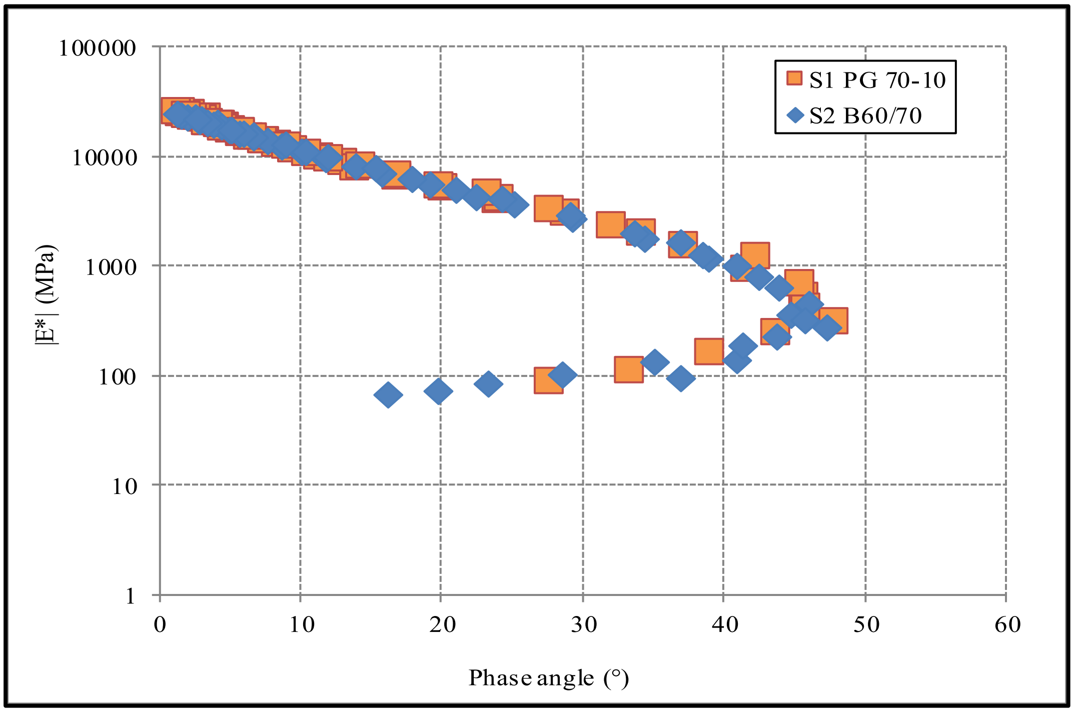


Figure 3. 12 Complex modulus results in Black Curve diagram

Figures 11 and 12 present the results of the complex modulus in Cole-Cole diagram. The x-axis explains the Storage modulus (E_1) and the Y-axis explains the loss modulus (E_2). At low temperatures, there is a decrease in the loss modulus and increase in the storage modulus. The results of Cole-Cole diagram show that there is a difference of modulus at low temperatures. This is also seen in Table 7 which shows the results of 2S2P1D model (refer to E_0), while analysis of the black diagram shows that the behaviors of mixes are almost the same at high temperatures.

These results may also be combined on a master curve. To do so, the frequency of each isothermal curve can be shifted to create a single master curve at one reference temperature. Figure 13 displays the master curves (complex modulus norm) for both mixtures. The top of the curve (at high frequencies and low temperatures) is the same for all of the conditioned and referenced mixtures. This indicates that, after all cycles, the maximum stiffness value of the mix remained constant. The base of the master curve (at low frequencies and high temperatures), which corresponds to the minimum asphalt mixture stiffness value, remained constant after all cycles as well.

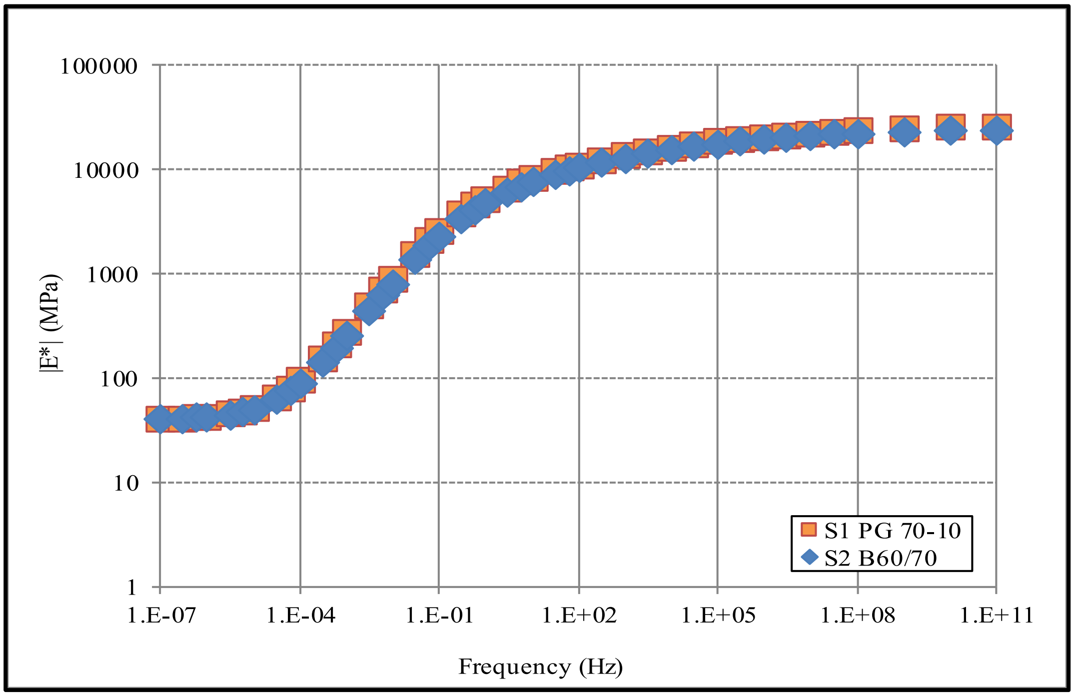


Figure 3. 13 Complex modulus results in master curve

The comparison results of the complex modulus of mixes can also be indicated on a master curve. Analysis of the Master curve in Figure 13 demonstrates that the results are almost the same at high temperatures (low frequencies) and intermediate temperatures.

3.4.4.2 2S2P1D Model Formula

The E^*master curves are shown in the 2S2P1D simulations for the asphalt mix test samples, as illustrated in Figures 11, 12 and 13. The model accounts for the data over a considerable range of temperatures and frequencies. The parameters for the 2S2P1D simulation are shown in Table 7 for all tested samples. (Olard & Di Benedetto, 2003), have shown that the parameters for the 2S2P1D model are reliable and that substituting a bitumen in the mixture maintains the values of the four constants of the model, namely k, h, δ, β. Additionally, the parameters are very similar between the two mixtures. In regards to the other parameters, , E_{00} is the glassy modulus (E^* when $\omega \rightarrow \infty$), and E_0 is the static modulus (E^* when $\omega \rightarrow 0$), which relates to the air void content and aggregate gradation. The Black space test and the Cole-Cole plane test will each display the results on a single curve.

Table 3. 7 2S2P1D model and WLF constant parameters for the mixes

Group	Specimen	V_a%	E_{00} (MPa)	E_0 (MPa)	k	h	δ	τ_E (s)
Mixture with PG70-10	S1	4.1	40	27,500	0.17	0.6	2.65	0.07
Mixture with B60/70	S2	4.6	40	25,000	0.16	0.6	2.65	0.07

Overall, these four tests show that the PG70-10 mixture with 33% natural sand performed better than the B60/70 with the same natural desert sand mix, in the hot and dry conditions of the countries under consideration.

3.5 Conclusions

This study reports the performance of an asphalt pavement mix containing a high percentage of natural sand and using PG70-10 as a binder. The mixture was evaluated in comparison with a mixture using B60/70 as a binder. The two mixes were evaluated primarily in terms of permanent deformation, i.e. rutting; for this, rutting analyzer and complex modulus tests were used to evaluate simplified viscoelastic continuum damage. The conclusion of this research can be discussed as follows:

1. The use of high-performance bitumen is critical to resist rutting in asphalt roads, due to the bonds it creates between particles that compensate for low particle angularity

2. This paper evaluates two mixtures using two different binders and indicates that PG70-10 is a more effective mixture to combine with natural desert sand for LVRs.
3. HMA with PG70-10 has a high level of stability and decreased flow, which leads to fewer instances of rutting and an improvement in the overall durability of the mixture.
4. The rutting analyzer test indicates that the mixture with B60/70 results in the most severe deformation and shorter life-span.
5. When the constant of the complex modulus at low temperatures and high frequencies is changed, there is a corresponding decrease in the loss modulus and increase in the storage modulus. In contrast, higher temperatures result in a higher stiffness value, and vice versa.
6. PG 70-10 makes it possible to incorporate a higher percentage of natural sand into an AC mixture.
7. The deciding factor is that, with the Superpave method, it is possible to use PG70-10 as a binder; this allows designers building LVRs in desert regions of developing countries like Libya to use low-cost local materials such as freely available rounded desert sands.

These results validate findings from previous research that studied the deformation of asphalt concrete pavement in Northern Africa and the Middle East.

3.6 Further work

Further research plans may explore adding polymers or fibers to improve the performance of the asphalt mixes. Other strategies include adding virgin asphalt binders to the mix, which could help the material meet performance measures beyond the temperature ranges of conventional binders. Information gained by these studies could allow engineers planning the next generation of road construction in hot and arid countries to improve the working life and cost effectiveness of these roads.

CHAPTER 4

EFFECTS OF NATURAL DESERT SAND ON PERMENET DEFORMATION BEHAVIOUR OF HOT MIX ASPHALT

Fathi S. Almadwi[1] and Gabriel J. Assaf[2]

Department of Civil Engineering and Construction, École de Technologie Supérieure (ÉTS), 1100 Rue Notre-Dame Ouest, Montréal, QC H3C, University of Québec, Canada.

Paper submitted to *the Journal of structural engineering and mechanics*, June 2019

Abstract

This study attempts to evaluate the permanent effects of using natural desert sand on the behaviour of asphalt mixtures. Previous studies have focused on the characterization of permanent properties of asphalts that uses natural desert sand, but few have done this in the Hot Mix Asphalt (HMA) itself. This study aims to determine the effect of using fine aggregate materials (natural and manufactured sand) in HMA, and how this affects the permanent deformation of the final HMA. The natural desert dune sand samples were collected from the desert in the hot and arid regions. The Index of Aggregate Particle Shape and Texture (IAPST), Marshall, the Rutting analyzer, and the Complex Modulus tests were undertaken to analyze and compare the properties of HMA containing two different types of bitumen binders, PG70-10 as a harder grade bitumen and B60/70 as a softer grade bitumen. The performance of mixtures containing the harder grade bitumen were found to be superior compared with mixtures containing the softer grade bitumen. These results are critical for engineers planning roadworks in hot and arid environments and allows for significant cost reductions by using locally available materials (i.e. desert sands) instead of more costly manufactured sands. With these findings, it is possible to partly use natural desert sands in HMA for low volume roads (LVRs).

Keywords: Natural Sand, PG70-10, B60/70, HMA, Rutting, Marshall, Rutting Analyzer, Complex Modulus Tests

4.1 Introduction

Hot Mix Asphalt (HMA) is the most commonly used construction material in road pavement. HMA made up of coarse and fine aggregates, filler and bitumen binder. The physical properties of the construction materials result in different interactions depending on how they are combined. This affects the mechanical character of HMA as well as the service life of the pavement. As such, an optimal selection of materials will result in superior pavement performance. Natural and manufactured crushed sand are both considered according to their physical properties and how these properties work together to affect the mechanical structure of a road pavement (Kallas, B. F., Puzinauskas, V. P., & Krieger, 1962). The greatest problem for road construction in remote desert areas is the lack of locally available quality aggregates, (Almudaiheem, 1990). A suggested solution is to substitute an amount of the fine aggregate in HMA with natural desert sand, (Almadwi & Assaf, 2017). HMA, mixed with the natural sand has a critical factor determining resistance to rutting is the selection of the high-performance bitumen, because the bonds it created between the particles helped compensate for the rounded particle of aggregate angularity (Almadwi & Assaf, 2017).

4.2 Background

Road construction, particularly low volume roads (LVRs) in desert areas, has been a serious problem because there is generally a shortage of locally available quality aggregates. The costs of bringing these aggregates from other regions makes it hard to keep roadwork projects on budget (Reza, Nayyeri, Niazi, & Jalili, 2011). This study assesses the potential to substitute a portion of the fine aggregate with natural desert sand in the hot mix asphalt (HMA). Laboratory tests were performed on dune sand powder (DSP) in a Portland cement mix, (Guettala & Mezghiche, 2011) but not on asphalt concrete. Some research in this area has used natural desert sand in various layers of the asphalt mix but there is no study on the effect of natural desert sand in HMA (Reza et al., 2011) and (Al-Juraiban & Jimenez, 1983).

Generally, in hot and arid environments, the most common reason for road failure is permanent deformation (rutting) and not fatigue cracking, (Al-Juraiban & Jimenez, 1983). Permanent deformation is caused by small instances of unrecoverable strain that accrue over time when axle loads are repeatedly made to the surface of the asphalt road. The permanent deformation can occur in the HMA, the base course, or in the subgrade, (Janoo V. and Korhonen C, 1999) and (Pérez & Gallego, 2010). The deformation is due to compacting processes under heavy tracking that result in volume changes, i.e. densification, and those that do not result in volume changes, i.e. shear deformation. This is exaggerated when the voids in HMA are either high or low (Sousa, Craus, & Monismith, 1991). It occurs each time a heavy truck applies a load on an asphalt pavement layer with inadequate shear strength. A higher pavement temperature increases the rate of rutting, (Chiranjeevi, T., Simhachalam, R., Kumar, D. A., & Raghuram, 2012) noted that the plastic flow that occurs in shear deformation was more common in the surface of the asphalt concrete than processes of densification. (Mahan, 2013) noted that there was more deformation in the surface asphalt concrete layer than in lower layers.

Aggregate weight in asphalt mixes is 90-95% and aggregate volume is 75-85%. The performance of an asphalt pavement, including its load-bearing capacity, as well as how easy the mix is to work with during construction depends directly on the physical characteristics of the aggregates used in the HMA, (Topal & Sengoz, 2005) and (Golalipour et al., 2012). Using manufactured (crushed, angular) aggregate results in a pavement that deforms minimally and that is less prone to deformations from compacting than the same mix made with aggregate from (rounded) natural desert sands (Kim ', Kim, & Khosla, 1992) and (Almadwi and Assaf, 2018). Studies on natural desert sand have shown that it is typically rounded and spherical (Reza et al., 2011) and (Marks, V. J., Monroe, R. W., & Adam, 1990). This is generally not as suitable for HMA as manufactured aggregate that is more angular and with more internal friction, and that consequently has a tendency to interlock.

When these reasons are considered in the context of remote, LVRs in natural sand desert regions (where dune sand is freely available), it is evident that a better understanding of how to use natural desert sand in HMA would be economically and practically useful.

This research demonstrates the effects of using natural desert sand with HMA in these areas. Three objectives are pursued. The first objective is to index and evaluate the shape of the aggregate particles, comparing the natural desert sand to crushed aggregate to determine the effect of the angularity of aggregate particles on HMA deformation. The second objective is to evaluate the proportion of the HMA aggregate mix containing natural desert sand. The third objective is to determine how the two binders (PG70-10 and B60/70) affect the performance of different mixes, by varying the ratio of natural desert to manufactured sands.

4.3 Materials and Laboratory Testing Procedures

4.3.1 Materials

Two asphalt binders were used in this study: Penetration Grade B60/70 and Performance Grade PG 70-10. Their properties are illustrated in Table 1. The diameter of all aggregates (manufactured crushed aggregate, natural sand, and filler) used in the study are between 0 and 5 mm. This study was used aggregate samples collected from the desert in hot and arid regions. This sand was tested to evaluate its engineering characteristics. In the collected samples, the sand was evaluated according to standards for particle-size distribution, (ASTM C 136, 2015). Samples were prepared using a mix of 63% crushed sand, 33% natural sand, and 4% of a mineral filler powder in the HMA mixes. Table 2 and Figure 1 show the grain-size distribution of all aggregates used in this study.

Table 4. 1 Asphalt binder properties

Binder Properties	Limits		Standard Specification	Samples Result	
	PG70-10	B60/70		PG70-10	B60/70
Specific Gravity	1.029	1.03	ASTM-D70 (T228)	1.0285	1.03003
Viscosity, Rotational, 135°C, Pa·s	≤3	-	ASTM-D4402 M-15 (T316)	0.394	-
Dynamic Shear, 70°C, G*/sinδ, kPa	≥1.00	-	ASTM-D7175 (T315)	2.20	-
Flash point, °C	≥230	≥235	ASTM-D92 (T48)	284	302
Mass Loss, %	≤1.00	≤0.2	ASTM-D113 (T240)	0.03	0.07
Creep Stiffness, 0°C, S, MPa	≤300	-	ASTM-D6 D6648 (T313)	102	-
Solubility, Min. %	≥99	-	ASTM-D4 D2042 (T44)	99.96	-
Penetration at 25°C, dmm	-	60-70	ASTM-D5 D4402 (T316)	-	64.7
Ductility at 25°C, cm	-	≥104	ASTM-D113 D70 (T51)	-	143
Softening point, °C	-	48-56	ASTM-D36 (T53)	-	51.7

Table 4. 2 Gradation of fine aggregate

Agg size (mm)	Passing (%)		
	Crushed Sand	Natural Sand	Filler
10.0	100.0	100.0	100
5.00	96.5	100.0	100
2.50	51.6	94.6	100
1.25	24.2	37.8	100
0.63	12.5	23.7	100
0.32	8.7	21.2	99
0.16	5.6	14.2	97
0.080	4.8	2.9	79

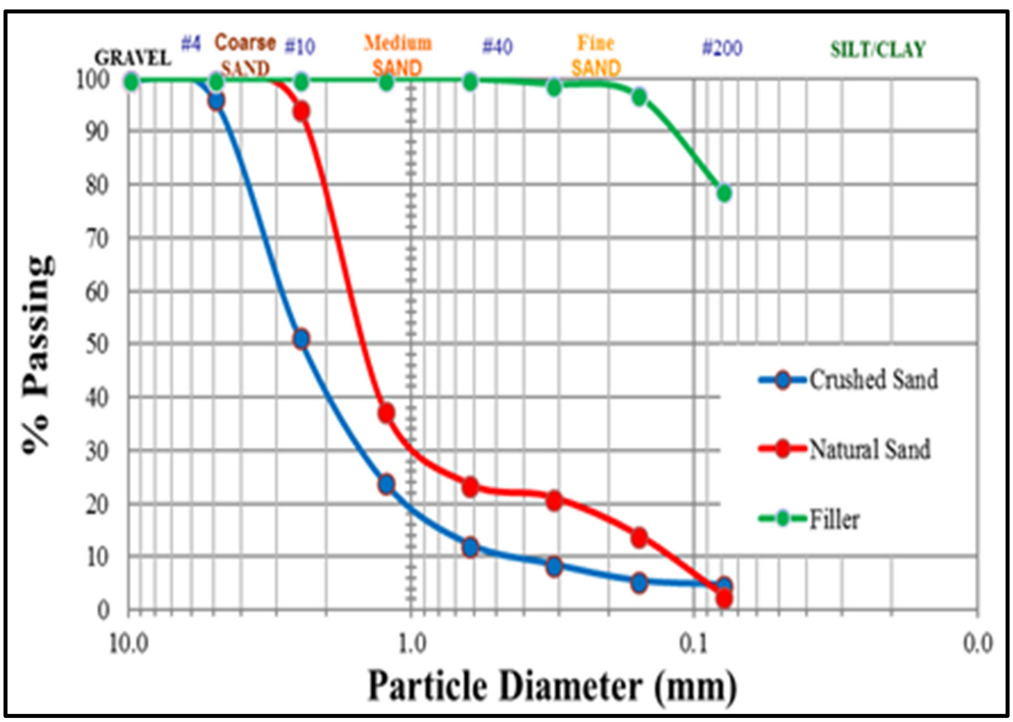


Figure 4. 1 Gradation of crushed and natural sands

4.3.2 Sample preparation

The preparation procedures were the following. The bitumen was brought to a fluid state by heating it in the oven to the recommended temperature of 162°C for PG70-10 and to the recommended temperature of 156°C for B60/70. The fine aggregates were also heated under the same conditions to eliminate the moisture content.

4.3.3 Laboratory Testing Procedures

4.3.3.1 Index of Aggregate Particle Shape and Texture (IAPST) test

IAPST helps to evaluate the effects of aggregate surface texture and shape on the strength and compaction properties of asphalt mixtures and soil aggregates. A standard aggregate particle has characteristics including surface texture, shape and angularity that all affect the final air void ratio and the rate of change in the void ratio when the aggregate is compacted into a mould (or is under vehicle load); this shows the need for particle indexing (Vincent C. Janoo, 1998). The indexing test was done with natural desert sand as well as the manufactured aggregate to find the roundness of the natural sand samples. The test evaluated the characteristics of HMA with regard to the two aggregates, according to (ASTM D3398, 2006a).

Normally, aggregate material is sorted into various size fractions before use, as shown in Table 3. In this project the aggregate was sorted into precise grades by using six sieves, each one slightly finer than the last, allowing particles of between five and zero mm to pass through successively finer sieves. This also helped eliminate the known effects of inter-sieve aggregate grading. The specific gravity of each aggregate size was determined according to (ASTM C127, 2012). The tamping rod and mould used for the aggregate sizes were according to (ASTM D3398, 2006a). These are shown in Table 3. The aggregate was packed into three layers, each of which was compacted with ten drops of a tamping rod, where each drop is from a height of 50 mms. Once the third layer was compacted, grains of aggregate were added to the surface to make it level with the outer edge of the mould. The filled mould was then weighed and the aggregate sample weight (M10) was measured with an allowance of 1 g. This procedure was done twice and the average M10 value was then used for the percentage air void

(V10) calculation. Likewise, air void percentage (V50) was established with a 50-drop procedure. The equations below were used to determine the IAPST for each given aggregate size:

Table 4. 3 Moulds and tamping rods details according to (ASTM D3398, 2006b)

Aggregate size (mm)		Mould details					Tamping rod details		
Passing	Retained	Type	Inside diameter (mm)	Inside height (mm)	Bottom thickness (mm)	Wall thickness (mm)	Diameter (mm)	Length (mm)	Mass (gm)
9.5	4.75	D	76.2	88.9	4.1	4.1	7.9	306	116
4.75	2.36								
2.36	1.18								
1.18	600 µm	E	50.8	59.3	3.8	3.8	5.3	201.7	34
600 µm	300 µm								
150 µm	75 µm								

$$V_{10} = \left[1 - \frac{M_{10}}{G_{sb(dry)} \quad . V}\right] \times 100 \tag{4.1}$$

$$V_{50} = \left[1 - \frac{M_{50}}{G_{sb(dry)} \quad . V}\right] \times 100 \tag{4.2}$$

$$I_a = 1.25\, V_{10} - 0.25\, V_{50} - 32 \tag{4.3}$$

where:

V_{10} = percent aggregate void once compacted at 10 drops for each layer.

V_{50} = percent aggregate void once compacted at 50 drops for each layer.

M_{10} = total average aggregate mass in the mould (g) once compacted at 10 drops for each layer.

M_{50} = total average aggregate mass in the mould (g) once compacted at 50 drops for each layer.
S = bulk specific gravity.
V = volume (ml) of cylindrical mould.
I_a = Particle index.

4.3.3.2 Marshall Test

The setup for the Marshall specimens was established by putting the mix in a preheated Marshall mould and then using a compaction hammer to compact it with 50 blows, for each of the two sides of the sample. Exactly forty samples of the asphalt mixtures were made with the bitumen binders under evaluation, where the bitumen content was tested at 4%, 4.5%, 5%, 5.5%, 6%, by weight of the aggregate mass, to find the optimum bitumen content, which was, in the end, 5.5%. Following this, the specimen's bulk specific gravity was tested, (ASTM D1188, 1998); following this, the air void percentage was tested. According to ASTM standards, the Marshall stability and flow were evaluated at 65^{0}C, (ASTM D6927-15, 2015). Figure 2 shows the Marshall testing machine (ETS-145).

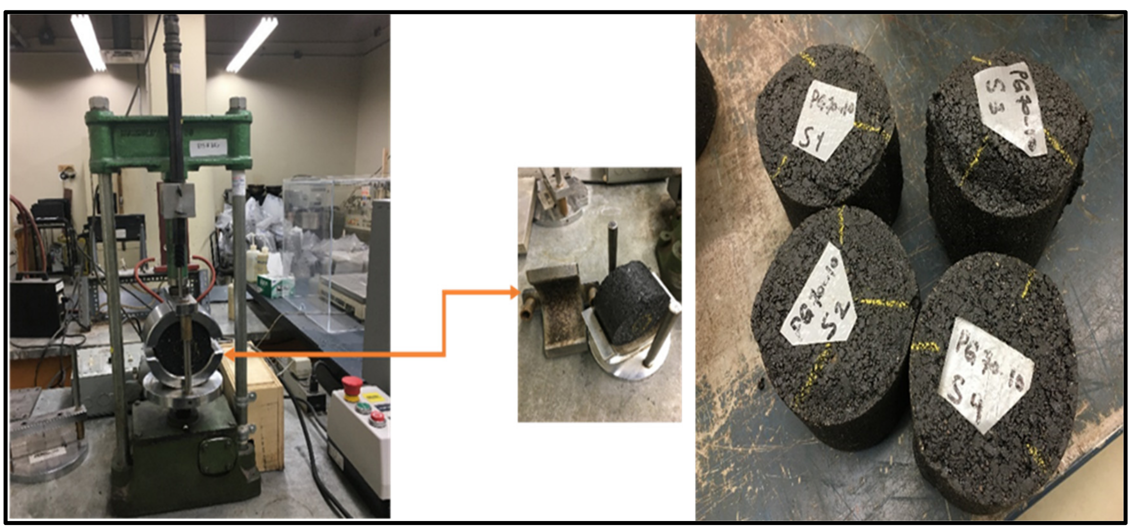


Figure 4. 2 Marshall testing machine and sample setup

4.3.3.3 Rutting Analyzer Test

The rutting test measured the resistance of samples to rutting according to Québec Standard LC 26-400, (Ministère-des-Transports-de-Quebec, 2016). After mix design tests and analyses, four slabs with dimensions of 500 × 180 × 50 mm3 were prepared and compacted using the French slab compactor, pictured in Figure 3. The test was conducted at 60°C and at a tire pressure of 600 kPa. Rutting tests were executed at intervals of 1000, 3000, 10,000 and 30,000 passes. After each stage, the rutting depth was recorded to determine the best mix allowing for high-temperature stability.

Figure 4. 3 French laboratory slab compactor LPC

4.3.3.4 Complex Modulus Test

The thermomechanical tests were performed with a servohydraulic press chamber (MTS 810, TestStar II, Eden Prairie, Minnesota) with an electronic system Figure 4 (Ministère-des-Transports-de-Quebec, 2016).

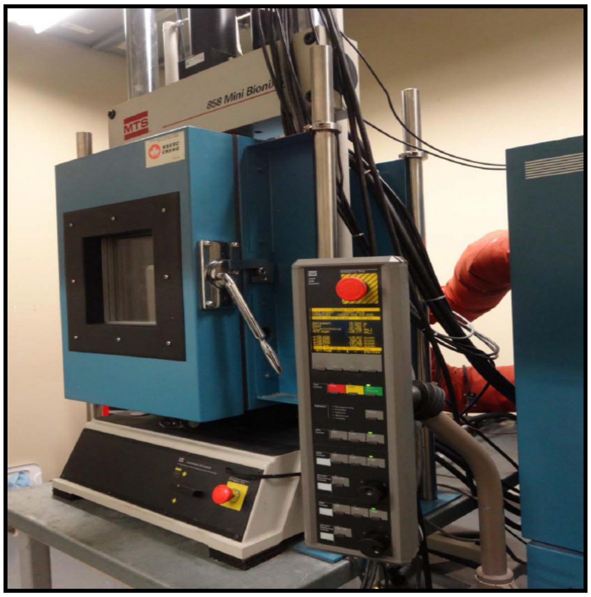


Figure 4. 4 MTS system and environmental chamber

According to the MTQ standards, LC 26-700 defines the complex modulus (E*) of asphalt mixes. E* determines the relation of stress and strain under sinusoidal loading. E* is the norm of the modulus explaining the test material stiffness, characterized by means of the dynamic modulus (Soenen, Redelius, & de La Roche, 2003).

$$|E^*| = \frac{\sigma_0}{\varepsilon_0} \tag{4.4}$$

where

$|E^*|$ is the norm of the complex modulus;

σ_0 is the maximum stress amplitude;

ε_0 is the peak recoverable strain amplitude.

The phase angle is represented by φ, which is a lag of the strain that creates the stress from the solicitation of the sinusoidal cycle. A phase angle of 0° would represent a completely elastic material, but a phase angle of 90° would indicate a completely viscous material. The sinusoidal stress of pulsation (ω), in terms of a ratio of the amplitude, is written as $\sigma = \sigma_0 \sin(\omega t)$, just as the sinusoidal strain's amplitude is written as $\varepsilon_t=\varepsilon_0 \sin(\omega t-\varphi)$, resulting in a steady state (Soenen et al., 2003).

$$E^* = \frac{\sigma}{\varepsilon} = \frac{\sigma_0 e^{i\omega t}}{\varepsilon_0 e^{i(\omega t-\varphi)}} \tag{4.5}$$

The viscoelastic characteristics of the materials needs to have a sufficiently wide range. As such, the samples were evaluated at eight temperatures ranging from −25 to 50 °C, specifically: −25, −15, −5, 5, 15, 25, 35, 50. The samples were then loaded at seven different frequencies, specially: 0.01, 0.03, 0.1, 0.3, 1, 3, and 10 Hz. These tests were executed under strain control settings. Through cyclic loading, the axial strain amplitude measured as 50 μ-strain.

4.4 Results of tests and discussion

4.4.1 Results of (IAPST) Test

(ASTM C127, 2012) and (ASTM C128, 2015) determine the standard properties of fine aggregates. Table 4 illustrates the test results of V_{10}, V_{50}, IAPST, specific gravity, and water absorption. The test indicate that natural sand results in a reduction in V_{10}, V_{50} and IAPST that corresponds with greater roundedness and smoothness of the aggregate texture and shape. The reason for this is that IAPST is an indirect measurement of aggregate packing density. Table 4 is best understood knowing that the manufactured crushed aggregate is more angular and therefore has a higher index than natural desert sand. The surface roughness of the crushed aggregate is obviously greater than the natural sand, the surface roughness of the crushed aggregate with a gradation of 0-5 mm is higher than the natural sand, which can affect its air voids and bulk density. Replacing the fine aggregate with a different shape and texture has led to a decrease in workability and compressive strength compared to round particle mixes. Therefore, the mixture having a different shape and texture has relatively better tensile properties, but lower properties of compressive strength and workability than a mixes with a round particle.

Table 4. 4 Crushed aggregates and natural sand properties

Aggregate type	Voids (V_{10})	Voids (V_{50})	IAPST	Specific gravity	Water absorption (%)
Crushed aggregate	46.64	44.38	15.20	2.720	0.11
Natural sand	41.37	39.86	10.20	2.634	0.33

4.4.2 Results of the Marshall Test

The Marshall test results are displayed in Table 5. Figure 5 shows that two mixtures have very similar Marshall stability results, although the mixture with PG70-10 is has a consistently higher stability measure than the mixture with B60/70; in both cases the optimum asphalt content is identical. Figure 6 shows that the flow results are different in both mix types, although the PG70-10 mixture has lower flow than the B60/70 mixture, which indicates that the PG70-10 will be more resistant to permanent deformations. It should be noted that the use of natural desert sand results in lower air voids in the compacted specimens, because of the fine sand fill the void between the particle due to increase the density and decrease the voids. A further consideration is that, with the B60/70 mix, where the flow is within acceptable limits (4% bitumen, 3.86mm flow) the air voids exceed the limitation (7.61%) and where the air void percentage is acceptable (4.5% bitumen, 6.94% air voids), it has too much flow (5.18mm). These issues are resolved with the PG70-10 and all results of the Marshall test are within acceptable limitations.

Table 4. 5 Marshall test results

Mixture ID	Bitumen Content (%)	Stability (Kgf)	Flow (mm)	Air Voids (%)	Optimum Bitumen Content OBC (%)
PG70-10 Mixture	4	13324	2.5	6.13	5.5
	4.5	16150	3.4	5.59	
	5	20029	3.7	4.37	
	5.5	23654	4	3.78	
	6	15187	4.21	3.21	
B60/70 Mixture	4	12033	3.86	7.61	5.5
	4.5	14620	5.18	6.94	
	5	17160	5.53	6.08	
	5.5	20285	6.10	6.21	
	6	12890	6.85	5.03	

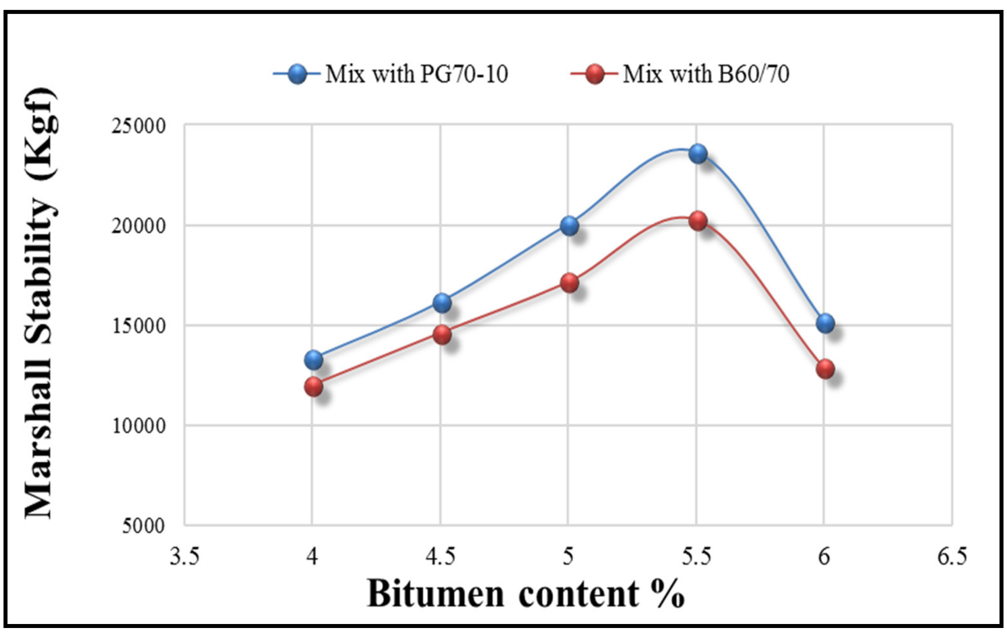


Figure 4. 5 Test results for Marshall stability using two asphalt mixes

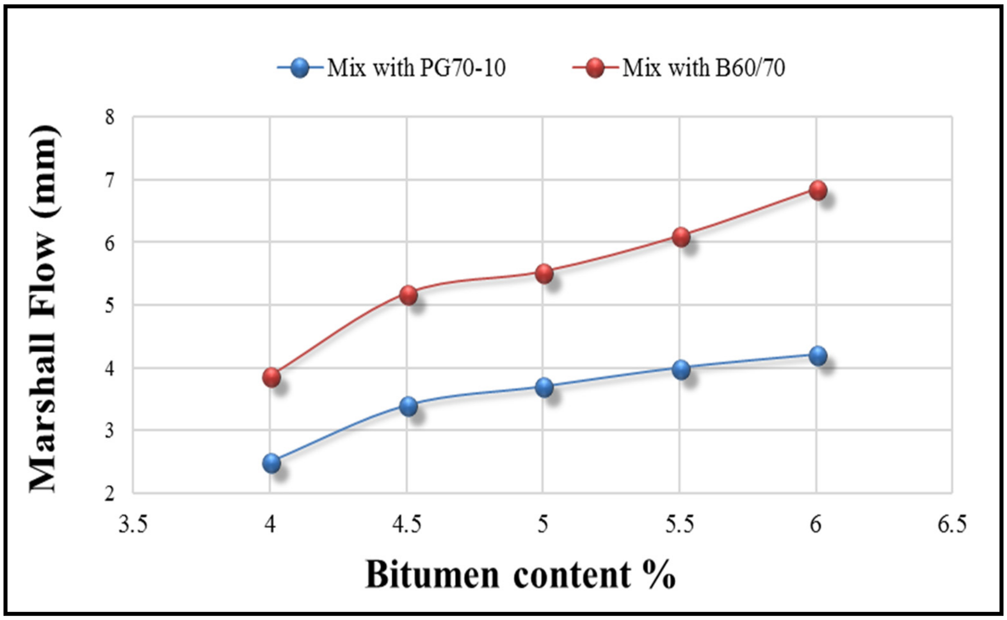


Figure 4. 6 Marshall flow test results for the two asphalt mixes

4.4.3 Results of the Rutting Analyzer Test

Figures 7 and 8 show standard test results of two asphalt mixture samples having an average rutting depth and calculated with regard to load pass measurements for both mixtures. Rutting depth at any given point is considered to be the difference found between the sample's maximum depression and the height of the adjacent shoulder of the depression. Rutting depth averages were calculated by taking the rutting depth at three different longitudinal points of the slab's central area, within a 50 mm range.

For the PG70-10 mixture, the test was ended once the maximum rutting depth attained about 6.11 mm after 30000 cycles. However, at the same number of cycles the test was terminated for the B60/70 mixture once the maximum rutting depth attained about 10.33 mm. These figures show that, following the initial period of rutting rate diminution, the rutting depth continues to increase in rough proportion to the quantity of load passes. The curve at these points can be characterized by a steady-state gradient. Following predictions, the rutting rate becomes greater in response to greater contact stress.

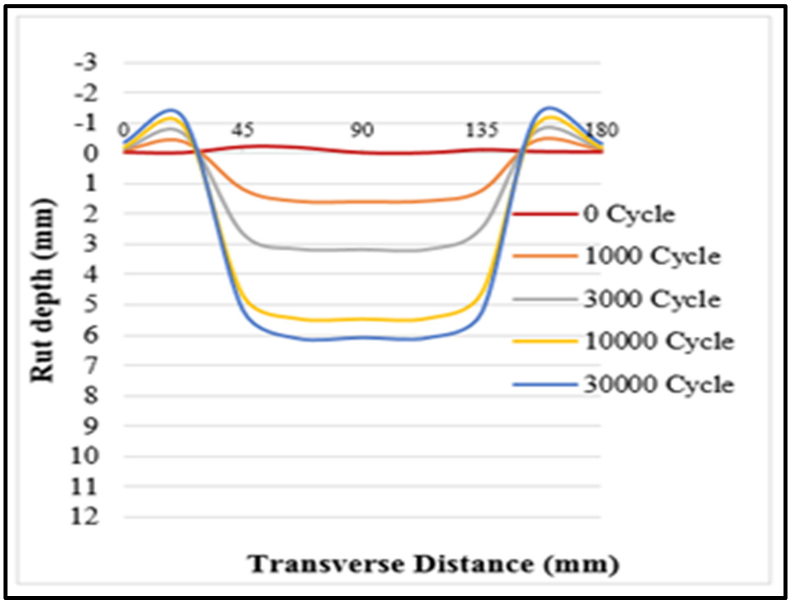


Figure 4. 7 Transverse rut profile (PG70-10 mix)

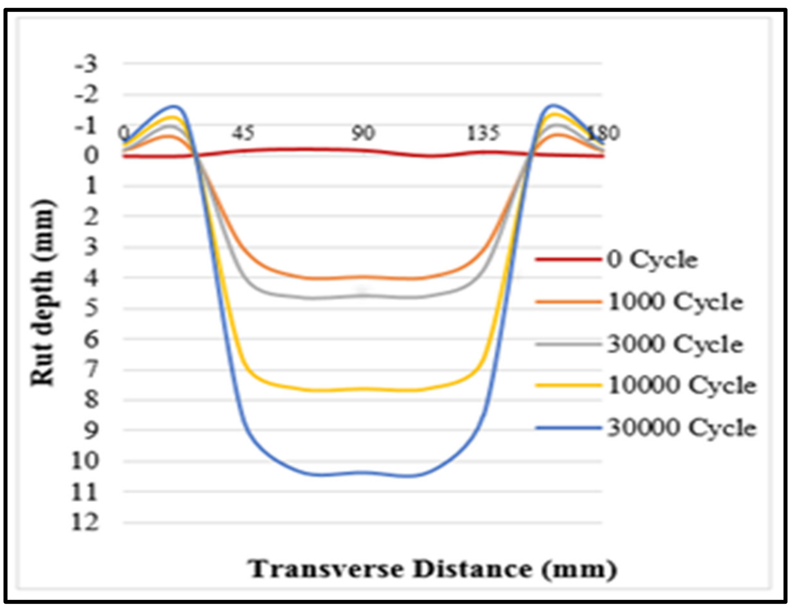


Figure 4. 8 Transverse rut profile (B60/70 mix)

The transverse rutting profiles from the central 50 mm of the samples were used to further analyze the initial performance at the point where the rutting rate started to diminish. These were then used for calculations determining approximate quantities of the asphalt mixture displaced from the depression and the quantity displaced to the shoulder. These calculations were calculated with the area ratio of the transverse rut profile (Collop & Khanzada, 2001):

$$R_{N+i/2} = \frac{A_{N+i}^{S} - A_{N}^{S}}{A_{N+i}^{D} - A_{N}^{D}} = \frac{\Delta A_{N+i/2}^{S}}{\Delta A_{N+i/2}^{D}} \tag{4.6}$$

where

$\boldsymbol{R_{N+i/2}}$ area ratio following N+i/2 load cycles;

$\boldsymbol{A_N^S}$ and $\boldsymbol{A_N^D}$ are the areas of material in the shoulders and the depression respectively after N loading cycles; and

$\boldsymbol{A_{N+i}^S}$ and $\boldsymbol{A_{N+i}^D}$ shoulder and depression asphalt mixture following N+i loading cycles.

From equation 6 it can be seen that, if the sample is compacted, shear flow is eliminated, $\Delta\boldsymbol{A_{N+i/2}^S}$=0 and $\boldsymbol{R_{N+i/2}}$=0. In contrast, if the material deforms at a regular volume $\Delta\boldsymbol{A_{N+i/2}^S}$=$\Delta\boldsymbol{A_{N+i/2}^D}$and $\boldsymbol{R_{N+i/2}}$= 1. When the sample dilates in the course of the deformation process $\Delta\boldsymbol{A_{N+i/2}^S}$ > $\Delta\boldsymbol{A_{N+i/2}^D}$ and $\boldsymbol{R_{N+i/2}}$> 1.

Figure 9 shows the results of Equation 6 when evaluated at an increment of ~1 mm of rutting depth. The area ratio is determined as a function of the quantity of the load applications in both asphalt mixes.

To minimize any errors that arise because of irregular deformation, the tests average three transverse profiles encompassing the central 50 mm of the slab; these are used in determining area ratios. Area ratios generally increase quickly at first, and go towards a constant ratio later in the testing process. For the PG70-10 mixture from 1000 cycles to 10000 cycles the area ratio generally increases quickly at first, from 10000 cycles to the end, where a constant value can be found. In the preliminary stage of the testing (1000 to 3000 cycles), the B60/70 mixture was relatively flat, with a greater slope from 3000 cycles to the end. Overall, Figure 9 indicates that, in terms of rutting, superior performance is observed with the PG70-10 mix, when compared with the B60/70 mix.

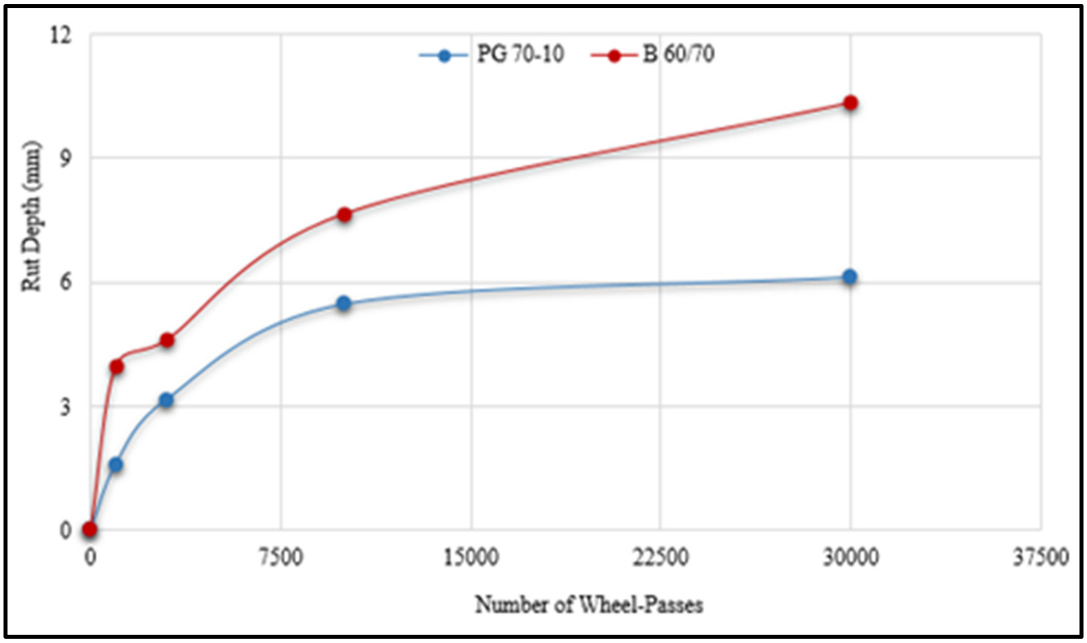


Figure 4. 9 Rutting analyzer test on both mixtures PG70-10 and B60/70

4.4.4 Results of the Complex Modulus Test

4.4.4.1 2S2P1D Model for the Linear Viscoelastic

The dashpot, spring and parabolic creep elements are used as a base for the 2S2P1D model. This is a typical model used in order to create a linear understanding of bituminous material function, including how such materials as mastics, mixes, and binders behave in viscoelastic, unidimensional, and tridimensional ways (Pouget, Sauzéat, Di Benedetto, & Olard, 2010) and (Mangiafico et al., 2014). The below equations describe the complex modulus, at a specific temperature:

$$E^*_{2S2PID} = E_0 + \frac{E_{00} - E_0}{1 + \delta(i\omega\tau)^{-k} + (i\omega\tau)^{-h} + (i\omega\beta\tau)^{-1}} \quad (4.7)$$

where

$\boldsymbol{E}_0$ the glassy modulus when $\omega \rightarrow \infty$;

$\boldsymbol{E}_{00}$ the static modulus when $\omega \rightarrow 0$;

$\boldsymbol{\omega}$ is the pulsation, $\omega = 2\pi f$ (f is the frequency);

$\boldsymbol{k}, \boldsymbol{h}$ constant such as $0 < k < h < 1$: δ constant;

$\boldsymbol{\beta}$ parameter linked with $\boldsymbol{\eta}$, the Newtonian viscosity of the dashpot;

Reference temperature

$$\mathrm{T_{ref}}, \tau_{\mathrm{E}} = \tau_{0\mathrm{E}} \tag{4.8}$$

In order to characterize the properties of 3D LVE for the bitumen, the constants $\boldsymbol{E}_{00}$, $\boldsymbol{E}_{0}$, $\boldsymbol{\delta}$, $\boldsymbol{k}$, $\boldsymbol{h}$, $\boldsymbol{\beta}$, $\boldsymbol{\tau}_{0\boldsymbol{E}}$ are used at the specific test temperatures. Equation 9 shows WLF law and this approximates evolutions in $\boldsymbol{\tau}_{\boldsymbol{E}}$, such that $\boldsymbol{\tau}_{\boldsymbol{E}}$ at temperature $\boldsymbol{T}_{\boldsymbol{ref}}$ is shown to be $\boldsymbol{\tau}_{0\boldsymbol{E}}$. The temperature effect once determined allows for the constants criteria become nine effects; as such, this represents the two WLF constants (which are indicated as $\boldsymbol{C}_{1}$ and $\boldsymbol{C}_{2}$, based on the reference temperature).

$$\log\left(\mathrm{a_T}\right) = -\frac{\mathrm{C_1}(\mathrm{T} - \mathrm{T_{ref}})}{\mathrm{C_2} + \mathrm{T} - \mathrm{T_{ref}}} \tag{4.9}$$

The master curves for E^{*} are seen in the 2S2P1D simulation for the HMA samples, as shown in Figures 10, 11,12 and 13. A wide range of temperatures and frequencies in the data can be described by the model. Table 6 shows the parameters used in the 2S2P1D model for all samples.

Table 4. 6 2S2P1D Parameters

Samples	E_{00} (MPa)	E_0 (MPa)	k	h	Δ	$\tau_E(\sigma)$	β
PG70-10 (S1)	75	29000	0,13	0,5	2,1	5,50E-01	250
B60/70 (S2)	70	26500	0,15	0,5	2,1	2,00E-02	200

As demonstrated in (Benedetto, H. D., Delaporte, B., & Sauzéat, 2007), the 2S2P1D model parameters are consistent. Replacing the bitumen in a sample allows the k, h, δ, β values to remain constant, as shown in the complex modulus tests were carried out on two test samples of two different bituminous mixes of the ESG5 type. The granular skeleton was the same for both bituminous mixes. The only variable between the two samples was the bitumen. The first mix used PG70-10 bitumen and it was labeled S1. The second mixture used B60/70 bitumen and was labeled S2. The experimental results of these two tests were modelled using the rheological model 2S2P1D and the results are presented below using the Cole-Cole curve and the Black space curve.

Figure 10 shows the results of the complex modules of the two samples that were tested with the Cole-Cole plot. There were no significant differences between the results of the two mixtures. However, in the very high frequency domain, the two mixtures displayed a slightly different behavior (Almadwi & Assaf, 2019).

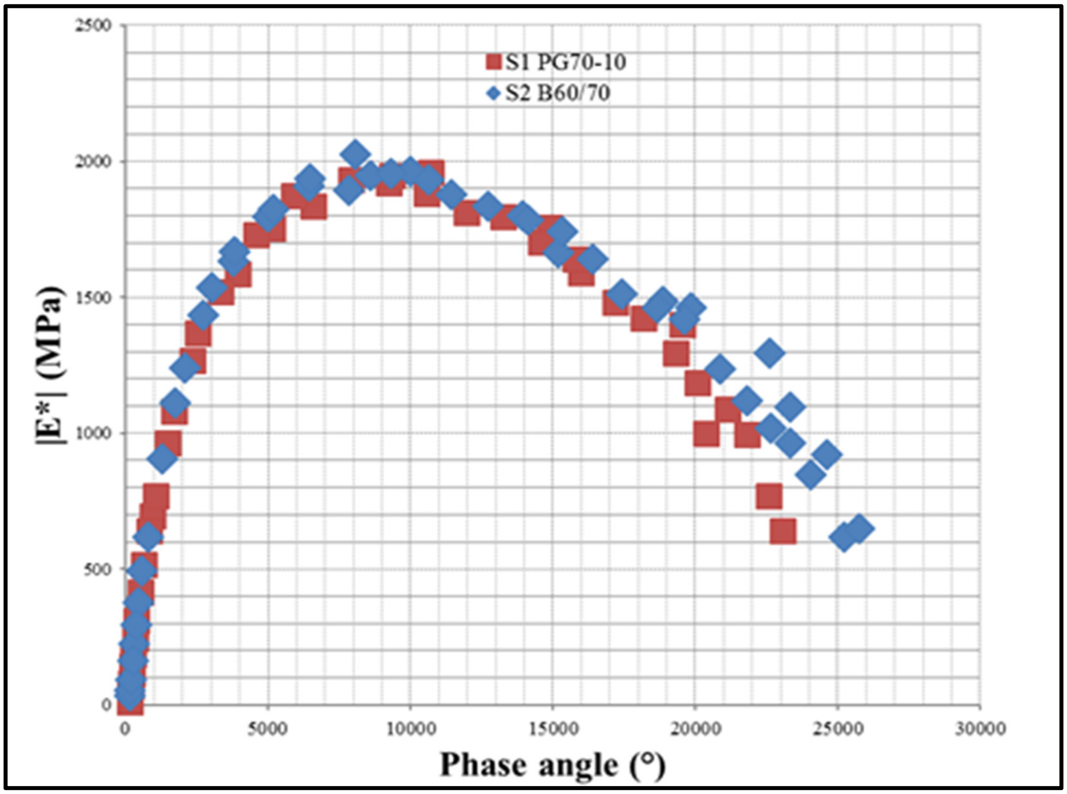


Figure 4. 10 Cole-Cole Diagram, Taken from (Almadwi & Assaf, 2019)

Figure 11 shows the results of the two tests in the Black space. Again, there was no difference between the phase angle values of the two bituminous mixtures (Almadwi & Assaf, 2019).

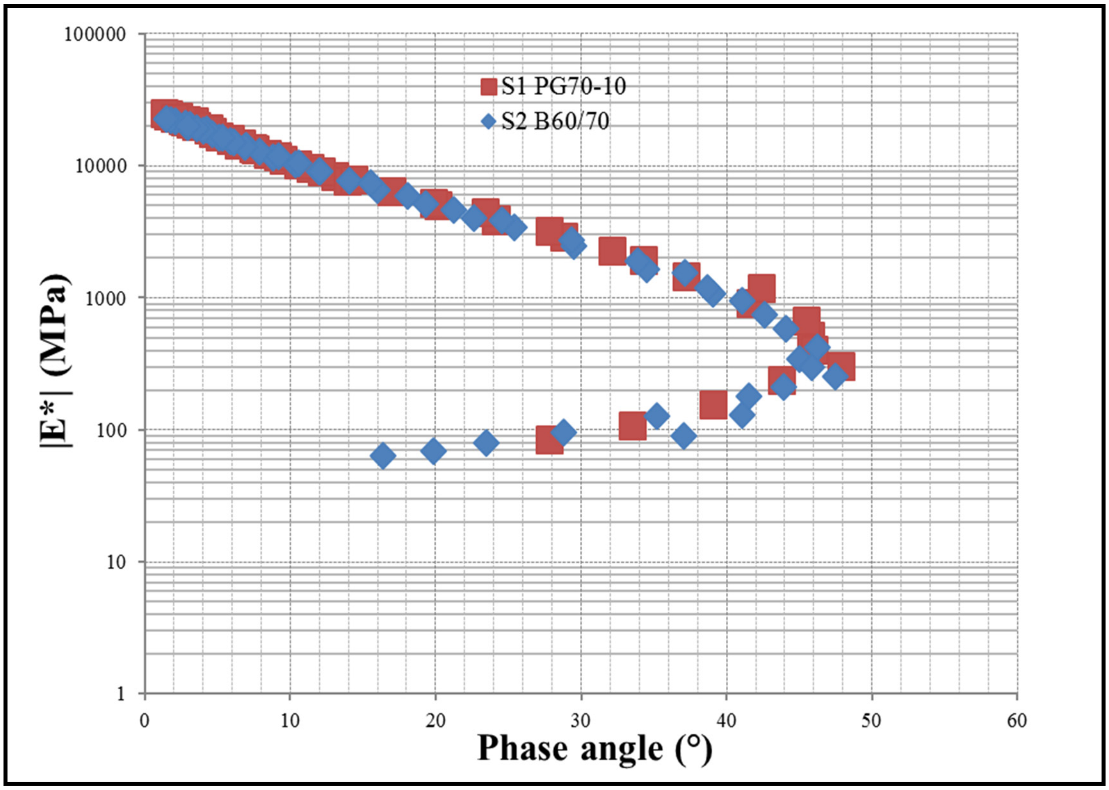


Figure 4. 11 Black Space diagram, Taken from (Almadwi & Assaf, 2019)

It is difficult and not sufficiently objective to make a direct comparison of the modules of the two bituminous mixes using the curves of the Cole-Cole plot and the Black space. To get a better idea of the differences between the experimental values of the two samples, we chose the S1 sample as a reference. Using a reference temperature of 15°C, the distances between S2 and S1 were calculated from the point of view of module and phase angle; the results are presented in the figures below.

Figure 12 shows the percentage differences between the experimental results of |E*| of S2 and S1 as a function of the S1 (a_T*f_r) equivalent frequencies. In absolute values, the differences ranged from 10% to 85%. The biggest differences were found in the low frequencies (85%) and the smallest difference was found in the high frequencies. It can be concluded from Figure 12 that the PG70-10 mixture in S1 had a higher modulus and therefore behaved better than the B60/70 mixture in S2, both at a reference temperature of 15°C.

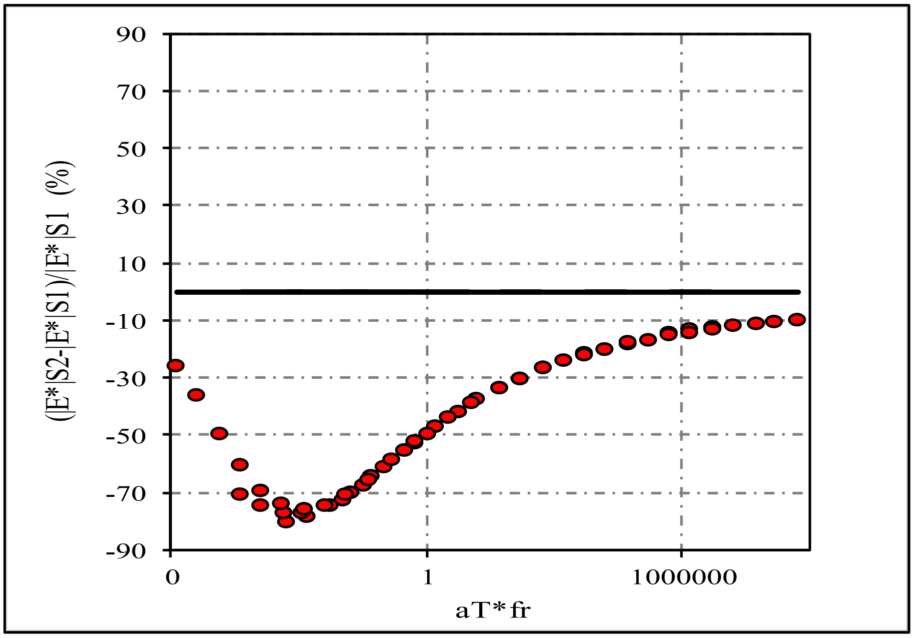


Figure 4. 12 Module differences between S2 and S1 as a function of equivalent frequency

Figure 13 shows the differences between the phase angles of the two mixtures S2 and S1. Contrary to observations made in the Black space, Figure 13 helps show the differences between the two mixtures. The low frequency results show that there are clear differences between the two mixtures. These differences in the high frequencies are not significant. It should be emphasized that the last two points in Figure 13 were not taken into account in the analysis of the results.

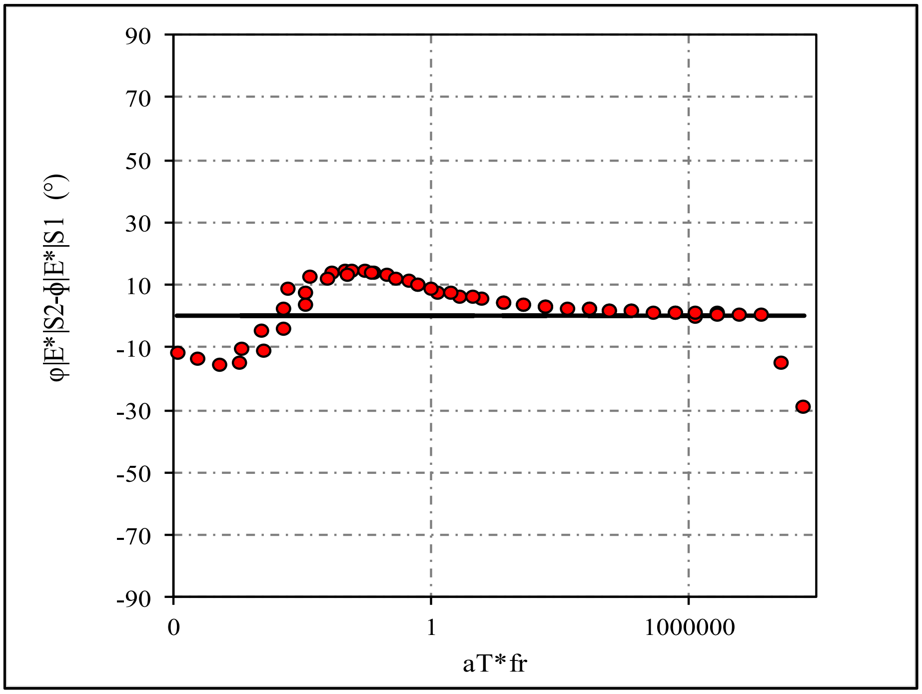


Figure 4. 13 Differences between phase angles as a function of the equivalent frequencies in S1

4.5 Conclusions

The main goal of this research is to determine the effect of using natural sand on the mechanical characteristics of the HMA using different binders. With the results obtained from the laboratory work, the following can be determined:

- Dune sand from deserts is naturally more worn down and rounded, i.e. has less angularity than similarly-sized manufactured aggregates.
- This research found the optimal percentage of dune sand that can be included in a long-lasting HMA for LVRs in hot and arid desert regions. The best ratio was determined as 33% natural dune sand. These results keep the quality of the final HMA within accepted Marshall Parameters.
- Selection of the high-performance bitumen was a critical factor determining resistance to rutting, because the bonds it created between the particles helped compensate for the lower particle angularity.

- The research evaluated two bitumen binders and found that the harder grade bitumen was superior if using naturally occurring desert sand for the described roadworks.
- The results of the rutting analyzer test show that the asphalt mix containing B60/70 has the most permanent deformation and the shortest in-use aging.
- PG70-10 mix rutting analyzer test results suggests that these mixes are subject to further compaction in the preliminary periods of the test; they are followed by steady-state deformations, a known results of material dilation.
- The use of natural dune sand, when used with the recommend bitumen, does not sizeably change the quality of the HMA.

4.6 Acknowledgments

The corresponding author gratefully acknowledge the great help provided by the staff of the road pavements laboratory at the Department of Civil and Construction Engineering at ETS (École de technologie supérieure) faculty of engineering of the Université de Québec, Canada, for their help with the lab work presented in this paper.

CHAPTER 5

EFFECTS OF BRICK POWDER ON THE PROPERTIES OF HOT MIX ASPHALT

Fathi S. Almadwi[(1)] and Gabriel J. Assaf[(2)]

[1,2] Department of Civil Engineering and Construction, École de Technologie Supérieure (ÉTS), 1100 Rue Notre-Dame Ouest, Montréal, QC H3C, University of Québec, Canada.

Paper submitted to *Journal of materials in civil engineering (ASCE)*, June 2019

Abstract

This study examines Brick Powder (BP) reclaimed from construction and demolition waste instead of using other fillers in a hot mix asphalt (HMA) that also uses readily available natural desert sand in the mix in hot and desertic areas. For this research, two HMA mixtures were designed using different fillers: one using BP and the control using limestone powder (LSP). The samples were subjected to the rutting, creep compliance, indirect tensile strength modulus (ITSM) and moisture susceptibility tests. The findings were that the mixtures using BP had superior mechanical characteristics to mixtures using LSP. BP showed a marked improvement on asphalt pavement performance and it resulted in a flexible road pavement with a long expected lifespan.

Keywords: LSP; BP; rutting; creep; moisture susceptibility; ITSM; HMA.

5.1 Introduction

Throughout the paving industry, petroleum-derived bitumen is essential. Certain properties of bitumen, including impermeability, viscosity but also its relatively limited costs have made it the most common binding material in the road pavement industry (García-Morales, M., Partal, P., Navarro, F. J., Martínez-Boza, F. J., & Gallegos, 2007).

Flexible pavements are commonly used in hot and arid areas. These pavements require great amounts of aggregates and mineral fillers for their construction and maintenance amounting to approximately 95% of the final mass of the asphalt concrete mix (ACM). As civil infrastructures grow, their development has led to a much greater need for the base materials, such as quarry rock and other natural resources. The need for substitutes is essential (Almadwi and Assaf, 2018). Libya is a prime example of what many countries in Africa or elsewhere have faced, i.e. they must cope with limited good materials and the marked rise in the quantity of waste from construction and demolition caused by changes in social and economic needs; these are often spurred by advances in the gross domestic product (GDP) of a particular country (UNDP, 2007), (Shiboub and Assaf, 2019). Following the events of the 2011 Arab Spring, Libya has had to deal with a much greater volume of these wastes and learn how to manage it. This waste has come from damage from the conflict. The waste generated since 2011 has been estimated to be approximately 82 million tonnes (Ali and Ezeah, 2017). Of this, much has possible application in construction if it were reused or processed. The recycling of fillers or aggregates would be both economically and environmentally preferable to using new materials (Ali and Ezeah, 2017).

5.2 Background

Bitumen has great susceptibility to temperature, where high temperatures cause rutting and low temperatures cause cracking. Bitumen is further limited in industrial applications by fatigue and aging. Heavy traffic loads and wide temperature variations are the leading causes of distresses on bituminous pavements. As such, modifying certain characteristics of bitumen becomes a worthwhile subject of research (Airey, 2003). Modifiers including polymers, sulfur, amine, carbon black, and fly ash have been tested and found to favourably change physical and flow characteristics of the hot asphalt mix (Sun & Lu, 2007).

In (Almadwi & Assaf, 2019), they compared the use of two bitumen types, taking as a control a fine aggregate mix, including 33% natural desert sand from the area. The mix using bitumen PG70-10 had superior properties to the mix using bitumen B60/70, given the application, i.e. low-volume roads in very hot and arid areas.

The use of crushed bricks as an aggregate is a promising area of research. The reports published to date indicate that crushed brick affects the mechanical performance of cement concrete. Nonetheless, it is not certain that this recycling of crushed bricks as an aggregate in concrete cement should be generalized (Huang, Y., Bird, R. N., & Heidrich, 2007) and (Debieb & Kenai, 2008).

In these applications, bricks from demolition waste are made into a powder. There are a number of ways to do this. One study in Japan exposed the bricks to extreme heat, turning the bricks into a slime ash; a Hong Kong study crushed the bricks to create filling material (Tam & Tam, 2006). In China, one study evaluated the mixtures prepared with BP and found that it had better mechanical properties than the mixtures with LSP (Chen, M. Z., Lin, J. T., Wu, S. P., & Liu, 2011). A study done in Italy researched the use of BP as a material to replace Portland cement; the research concluded that the mortar made with BP had good performance characteristics (Corinaldesi, V., Giuggiolini, M., & Moriconi, 2002).

Another group examined the use of BP from waste materials as a replacement for some elements when making cement mortar; the study found that there was good potential application of BP for making pozzolanic cement (Naceri & Hamina, 2009).

Nonetheless, until quite recently, there has been little research on the use of BP in HMA. Other waste materials, however, have been studied as possible powder mineral fillers in HMA. One study (Do, H. S., & Mun, 2008), looked at the use of recovered lime from waste as a possible filler in HMA; the conclusions suggest that lime improves HMA properties such as stiffness, fatigue resistance, permanent deformation and the overall lifespan of an asphalt pavement. Another study examined what effect cement dust had as a filler on an HMA's mechanical characteristics; the study showed that cement dust from construction waste can be substituted completely instead of LSP in HMA (Hassan Y. Ahmed, 2006). Fly ash has also been researched as a replacement filler and has been found to be effective in the wearing course as a filler (Tapkın, 2008). According to the above research, these waste powders can be used as filler in HMA with no negative consequences and, in many cases, with improved engineering properties. The above suggests that using recovered bricks as a filler for HMA could be a worthwhile subject of study, i.e. it has similar mechanical characteristics as the above powders and, because it is derived from waste materials, may be a relatively cheap solution to construction needs while also making use of construction and demolition waste. This environmental aspect cannot be understated due to pressures on raw resources and also on disposal facilities (Ma, F., Sha, A., Lin, R., Huang, Y., & Wang, 2016). Other materials that are being studied are glass powder and rubber powder (Bilondi1a, M. P., Marandi, S. M., & Ghasemi2b, 2016); like BP, these are being considered for their environmental properties as well.

The present paper is a study of BP, focusing on its mechanical characteristics as a mineral filler in HMA; tests including Rutting, Creep Compliance, Moisture Susceptibility and Indirect Tensile Strength Modulus (ITSM) tests are conducted to determine HMA properties. The results of these tests are used to analyse the viability of using BP as a filler in HMA; the BP is compared with standard LSP fillers.

5.3 Materials and test procedures:

5.3.1 Materials

5.3.1.1 Aggregate

The samples of desert sand for this research was taken from the sand dunes in southwestern Libya. This desert sand was mixed with crushed manufactured sand, having particles of between 0-5 mm. This study used HMA, where the nominal maximum size of fine aggregate was 5mm, which is normally called semi-grouted asphalt (ESG5); this is the standard for surface layers according to the Ministère des Transports de Quebec (MTQ). The basic physical characteristics of this composite aggregate is shown in Table 1.

Table 5. 1 Physical properties of aggregates

Aggregate Properties	ASTM Designation	Values
Water absorption (%)	C 128	0.87
Specific Gravity (g/cm^3)	C 128	2.736
Absorption loss (%)	DC 131	12.2

5.3.1.2 Filler

The clay bricks used for the BP were recovered from building waste collected in Libya since 2011. The recovered brick was cleaned and oven-dried at 110˚C for eight hours. Then, the brick was mechanically crushed and pulverised for 15 minutes until it became BP. LSP filler was the control mix. Physical filler characteristics are shown in Table 2. Chemical characteristics of the fillers are detailed in Table 3. Table 3 shows that CaO, SiO2 and Al2O3 are standard components of these fillers; LSP has a higher amount of CaO and BP has a higher amount of SiO2.

Table 5. 2 Physical characteristics of the BP and LSP

Property	LSP	BP
Passing (%) 0.63 mm	100	100
0.315 mm	97.6	99.2
0.16 mm	95.3	96.8
0.080 mm	92.5	93.4
Specific Gravity (g/cm^3)	2.775	2.723
Water absorption (%)	0.8	1.0
Hydrophilic coefficient	0.7	0.5

Table 5. 3 Chemical contents of the BP and LSP

The property	Total of Content %	
	LSP	BP
Silicon Dioxide (SiO_2)	13.07	72.5
Aluminum Oxide (Al_2O_3)	0.52	13.18
Iron Oxide (Fe_2O_3)	0.50	5.82
Sulphur Trioxide (SO_3)	0.06	0.20
Calcium Oxide (CaO)	48.0	2.0
Sodium Oxide (Na_2O)	0.09	1.16
Potassium Oxide (K_2O)	0.13	2.66
Magnesium Oxide (MgO)	2.88	1.46
Loss of ignition (LOI)	34.73	0.75
Total	99.98	99.83

The BP and LSP data in figures 1, 2 and 3 have been measured with a Scanning Electron Microscope (SEM). In the images from the SEM, the surface area of the BP particles was considerably less even than LSP; the distribution of the particles was also more homogenous than the LSP particles. These factors suggest that the BP should have a higher rate of water adsorption than LSP.

Figure 5. 1 The BP and LSP

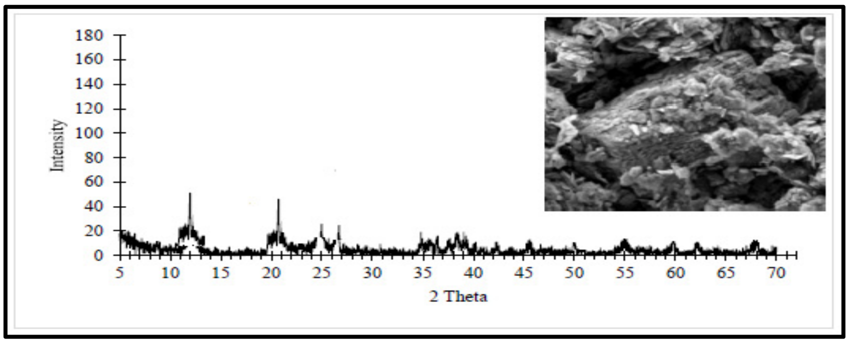


Figure 5. 2 BP morphology under the SEM and XRF

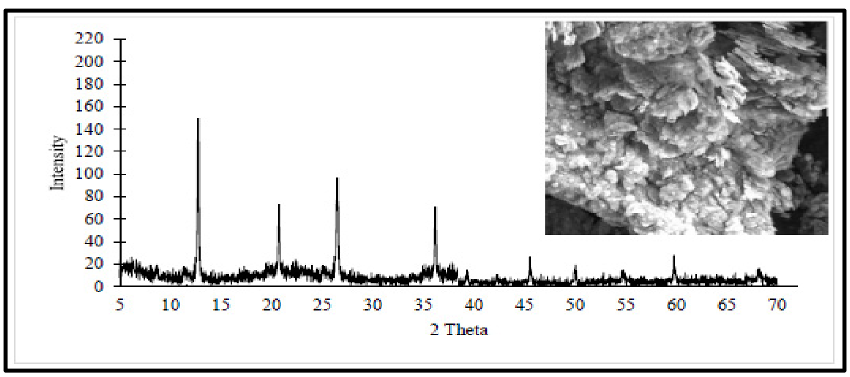


Figure 5. 3 LSP morphology under the SEM and XRF

The AASHTO T-305 drain down and tests on the specific surface area (SSA) were used to evaluate BP absorption. Drain down tests can indicate the amount of bitumen in the mixture. A lower drainage value means that there is a higher rate of absorption from the powder, either BP or LSP. Where there is a higher SSA, there is a greater absorption by the fillers. The test results are given in Table 4. As shown, BP has higher absorption in these tests than LSP.

Table 5. 4 Drain down and specific surface test results

Mixture ID	Drainage	Specific Surface cm^2/g
BP	$2.4 * 10^{-3}$	504.36
LSP	$1.7 * 10^{-3}$	451.7

5.3.1.3 Asphalt binder

One bitumen binder used in this study is Penetration Grade B60/70. Table 5 shows the physical characteristics of this binder. The asphalt pavement layer structure is typically constructed of, at minimum, two layers of different thicknesses, including surface binder layer, and base layers.

Table 5. 5 Physical properties of bitumen

Property	Criteria Range Limits	Specification	Lab Test Results
	B60/70		B60/70
Specific Gravity	1.03	ASTM D-70	1.03003
Flash point, °C	≥235	ASTM D-92	302
Mass Loss, %	≤0.2	ASTM E595	0.07
Penetration at 25°C, dmm	60-70	ASTM D-5	64.7
Ductility at 25°C, cm	≥104	ASTM D-113	143
Softening point, °C	48-56	ASTM D-36	51.7

5.3.2 Mixture design and asphalt preparation:

In this study, the Libyan road construction specification (No. 49/2002) for HMA that pertain to aggregate grading curves were used. Figure 4 shows the sieve gradation curves that were derived from the tests on the composite aggregate. Two filler types were used for mixes having the comparable aggregate gradations. The optimum bitumen content (OBC) was determined using the Marshall test to find the best bitumen ratios for the mixtures. An OBC of 5.5% (by aggregate weight) was found to be best for the two asphalt mixtures of BP and LSP; the optimum mineral filler mix of 4% (by weight of aggregate) was found to be the best for the two asphalt mixtures of BP and LSP.

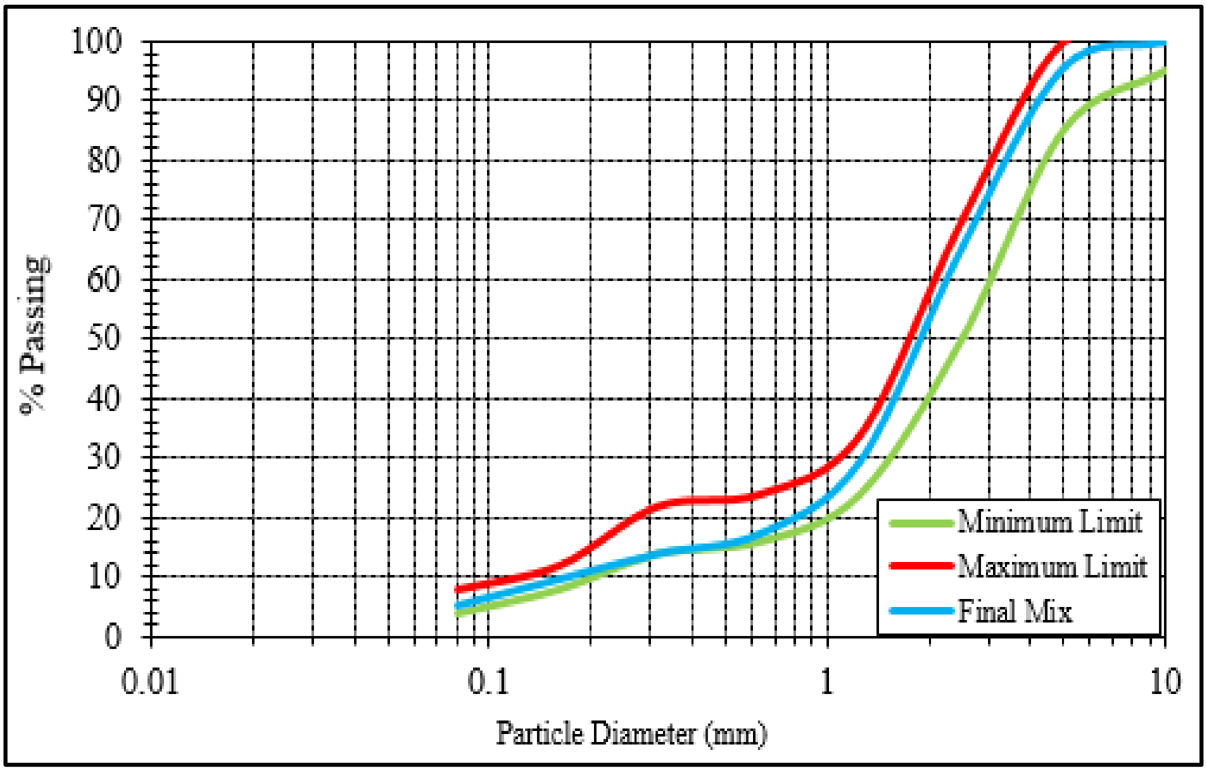


Figure 5. 4 Aggregates Gradation

5.3.3 Laboratory procedures:

5.3.3.1 French Rutting Tester (FRT)

In France, for over 15 years now the FRT has been used to evaluate rutting in HMA. FRT involves running a loaded pneumatic tire repeatedly back and forth across an HMA sample. In this way, the FRT can evaluate a sample's resistance to permanent deformation according to

AASHTO TP 63. Tests are usually conducted on slabs of 500 x 180 x 20-100 mm that have been compacted by a plate compactor; this setup can be seen in Figure 5. Generally, the tire is passed over a sample for 30,000 cycles while a 500 N load is applied to the tire, inflated to 600 kPa. The test chamber is usually heated to 60˚C. FRT is a primary test for estimating good and poor rut performance in the field.

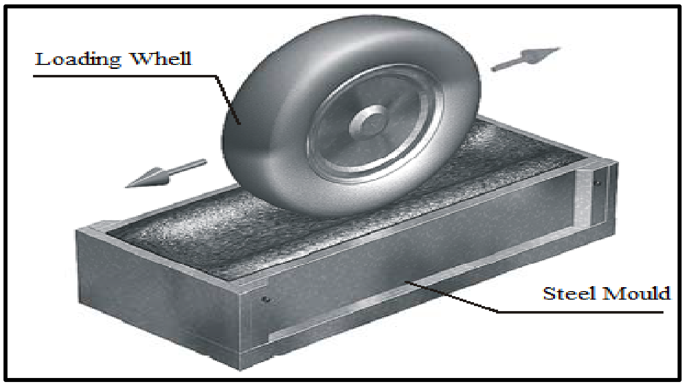


Figure 5. 5 Scheme of rutting test (FRT)

A test was done for each of two HMA sample that have the two different mineral fillers (BP, LSP); each test-run comprises two samples (S1 and S2, i.e. left and right); each mineral filler test was done twice. Therefore, there were a total of four reference specimens for each of BP and of LSP. The Québec Standard LC 26-400 (MTQ 2016) as well as the French HMA specifications for test samples are shown in Table 6.

Table 5. 6 Specifications for the FRT

Layer Thickness	**Layer Type**	**Cycles Number**	**Max Rutting %**
	Subbase Base	10,000	≤ 10
6-8 cm	Base Course Wearing Course	30,000	≤ 10
3-4 cm	Wearing Course	1,000 3,000	≤ 10 ≤ 20
8-10 cm	Base Course (High modulus for rut resistance)	30,000	≤ 8

5.3.3.2 Indirect Tensile Stiffness Modulus test (ITSM)

ASTM D 4123 Standards and the EN 12697-26 ITSM were followed. According to the EN 12697-26 standard, the size and mass of the samples were determined using Eq. 1:

$$S_M = \frac{F\ (R + 0.27)}{LH} \tag{5.1}$$

where

S_M represents the Stiffness Modulus (MPa);

F pertains to the applied vertical load's peak value (N);

L is the horizontal deformation amplitude from the load cycle in mm;

H is the cylindrical specimen's mean thickness in mm;

R is the Poisson's ratio (-).

Figure 6 shows the MTS device used to perform the tests under conditions controlled for strain. The test takes specimens that are 63.5 millimeters thick and 101 millimeters in diameter. Testing temperatures for the stiffness modulus of the samples were 5, 25, and 40˚C. This was done so as to help see how temperature affects the mix. The Poisson ratios of 0.3, 0.35 and 0.4 were established for the above temperatures. Cycle periods of 3 seconds, 2 seconds, 1 second were based on the ASTM D 4123 typical loading pulse. The rise time (time wherein the applied load goes from zero to a maximum rise value) was 124 milliseconds. Nevertheless, the focus in the ITSM test is on the rise time because the evaluation of the Modulus is with respect to the full vertical load force; this value was responsible for the controlled deformation. As such, to choose the test values, it is practical to estimate a fourfold time increment pertaining to the rise time; this can be considered the effective cycle period.

Each mixture had a modulus of three samples that were each tested and from which an average was taken and used to present the results (18 samples tested totally). The Stiffness Moduli (SM) were chosen in laboratory trials at the temperatures of 5, 25, 40°C. The asphalt mixture temperature was analysed in reference to this fourfold rise in time with the horizontal deformation of 5 μm, where the rise times were 50 milliseconds, 100 milliseconds, 124 milliseconds and 150 milliseconds. These time rises were used in the tests and corresponded to the frequencies of 1.5Hz, 2Hz, 2.5Hz, 5Hz. Throughout, the pulse repetition maintained a interval constant of 3 seconds.

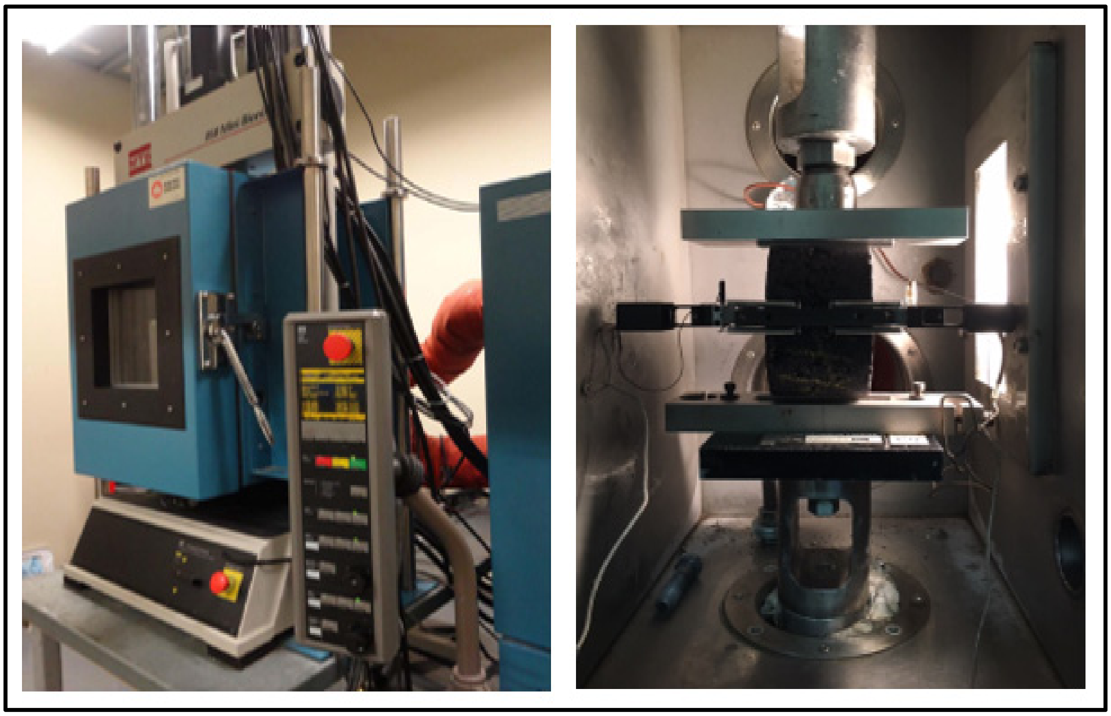


Figure 5. 6 MTS measurement system and environmental chamber

5.3.3.3 Moisture susceptibility test

The water sensitivity test (AASHTO T283) test is used to determine how sensitive an HMA sample is to water. Moisture damage results from the following factors: cohesion loss, and adhesion loss. Cohesion loss happens when the asphalt concrete mastic softens. Adhesion loss happens when there is water between the bitumen and the aggregate, causing the bitumen film to be stripped from the aggregate. The two mechanisms have a close connection and pavement that has been subject to moisture may show deformation due to a combination of both of these mechanisms.

The protocol involves compressing samples to an air void percentage of 7%. The cylindrical samples start with a 101.6 millimeter diameter and a 63.5 millimeter height, as shown in Figure 7. As a control, three specimens are tested without moisture conditioning; another three are conditioned by 80% saturation followed by a cycle of freezing (-15˚C x 16 hours); after this, they are soaked in a 60˚C warm water for 24 hours. These samples are then subject to an Indirect Tensile Strength (ITS) test where they are loaded at 25˚C under vertical pressure at a steady rate of 50.8 millimeters per minute and the force required to create permanent deformation in the sample is recorded. The ITS of the conditioned and controlled samples is analysed to determine the Tensile Strength Ratio (TSR).

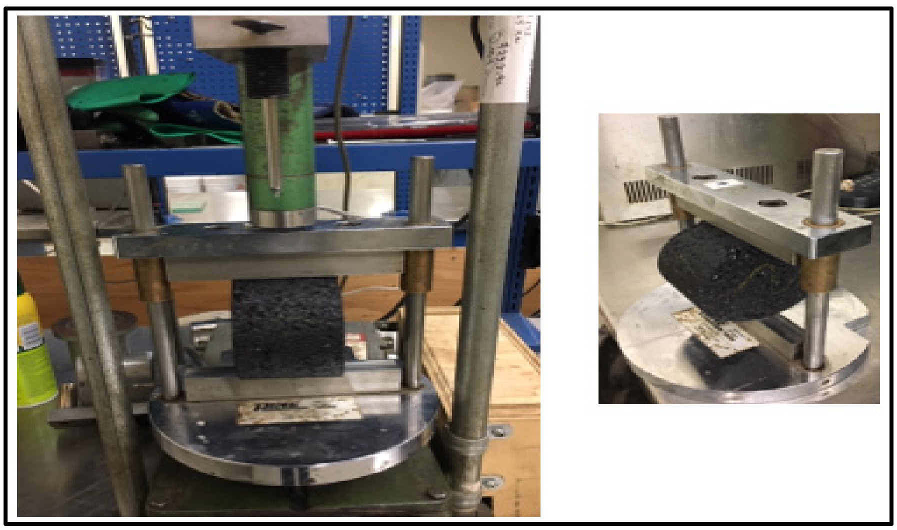


Figure 5. 7 ITS measurement system and sample setup

The sensitivity to water of the HMA can be determined with the TSR value in Eq. 2:

$$TSR = 100(T_{cond}/T_{uncond}) \tag{5.2}$$

where,

T_{cond} is the ITS average of the wet sample;

T_{uncond} is the ITS average of the dry samples.

The ITS and E-module are determined by Eqs. 3 and 4:

$$ITS = \frac{2 * P}{(\pi * h * d)} \tag{5.3}$$

where,

ITS Indirect tensile test (N/mm^2);

P Maximum test force (N);

h Height of test sample (mm);

d Diameter of test sample (mm).

$$E = \frac{8 * ITS}{(3 * \varepsilon_1 + \varepsilon_2)} \tag{5.4}$$

where,

ε_1 Vertical strain;

ε_2 Horizontal strain.

5.3.3.4 Creep tests

Creep is the deformation over time of viscoelastic materials subject to a loading force. Viscoelastic materials (e.g. plastics) can demonstrate rheological (flow) qualities as well as their viscoelastic qualities. This is also frequent in HMA. Such rheological qualities, when subject to applied load, result in creep. The creep test is a very standard test used to measure resistance to rutting. The creep test is a means of predicting the rut depth that can be caused by permanent deformation in layers of asphalt. There are a number of modes used in creep tests, including unconfined/confined, incremental/static, static/cyclic, but the general methods are approximately the same (Khiavi & Mansoori, 2017). The results of these tests are shown in Figure 8 in the strain-time graph; it is separated into the following aspects: initial, final, total, and reversible.

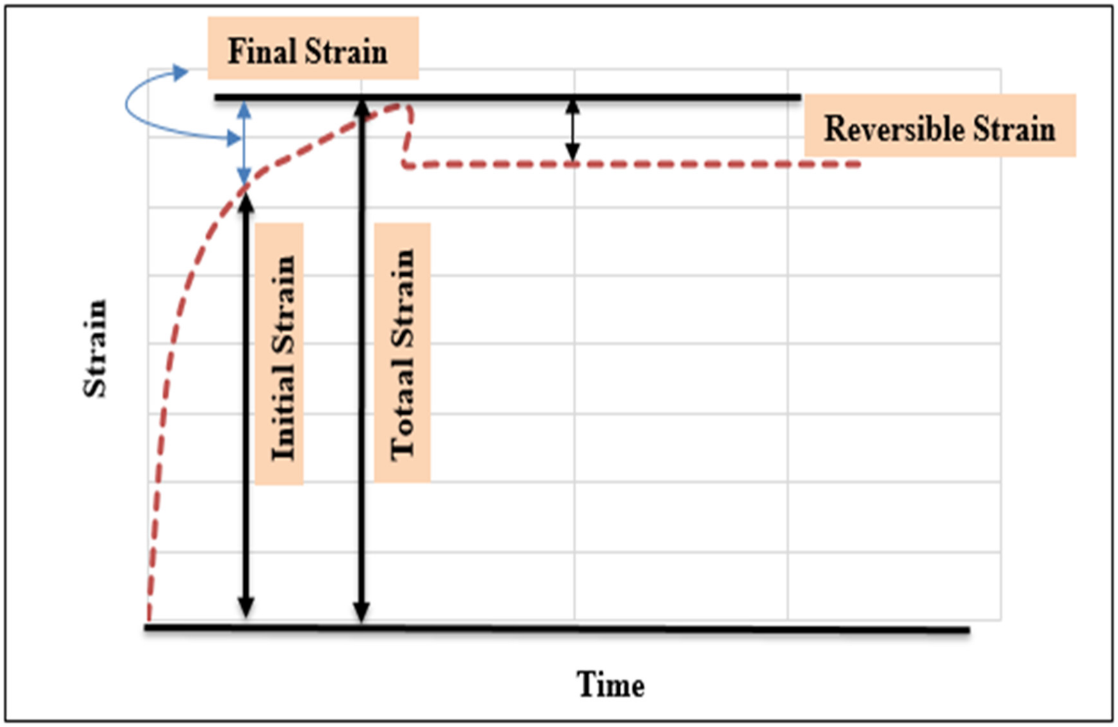


Figure 5. 8 Schematic division of strain components as measured by the creep test

5.3.3.4.1 Creep compliance

In this research, the T 322-07 creep compliance test was conducted with MTS device test equipment. To prepare the samples, that were 50 mm high with a diameter of 150 mm, they were mixed with a 7% air void percentage for each mix preparation. The creep testing was conducted at the following test temperatures: 0, 20, and 40°C for 1000 seconds each. For each mix, three identical samples were mixed and tested, and the displacement gages were fixed onto the surface of the samples to measure forces in vertical as well as in horizontal directions. These gauges were attached with a cyanoacrylate adhesive and with glue; the gauge placement was determined so that the extensometers could be suspended between sets of adapters, placed in opposition to each other on the specimen, always controlled for temperature. At that point, a static load was added to the diametral axis for a set period, most often 100 seconds.

The tests for creep compliance require a controlled load that does not pass the HMA's upper linear-elastic boundary, which most often is 500 microstrain; as such, it is non-destructive and each sample can be reused and tested at more than one temperature. Nonetheless, the test load needs to be high enough to create horizontal deformation; this deformation must be ≥0.00125 mm or 33 microstrain based on a 150 mm gauge diameter, thereby ensuring that any noise created in the process of data collection is not significant. Over the course of the load period, the deformations of both horizontal and vertical can be recorded; these measures are taken from two parallel faces of the samples that were sawn from a larger sample. In total, four extensometers were used, two attached to each face. Figure 9 shows the setup for the creep compliance test.

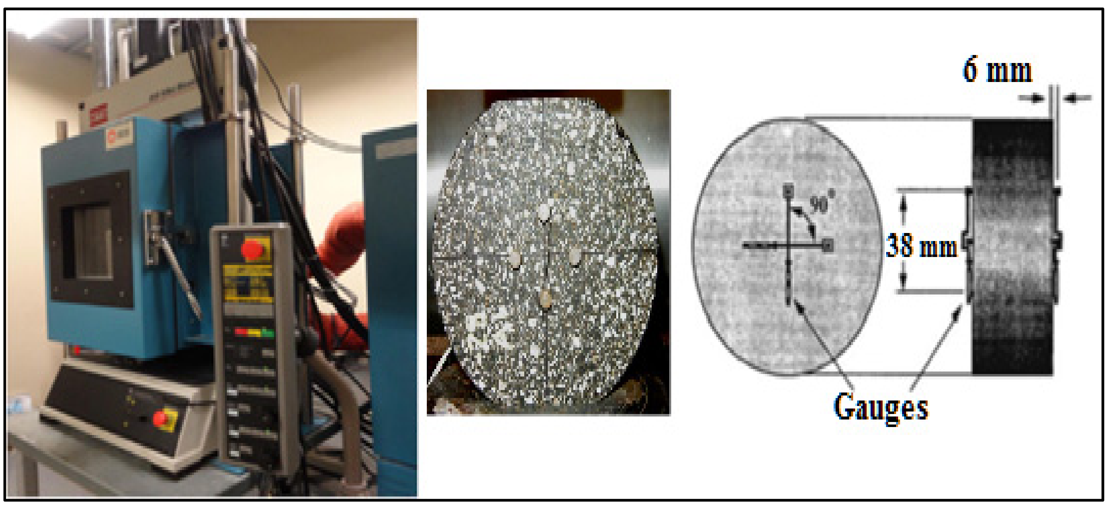


Figure 5. 9 Creep compliance test system

To calculate creep compliance, the following values must be put into Equation 5 below: the measure of the vertical and horizontal deformation, the gauge length of this measurement, the sample's size, the static load's magnitude.

$$D(t) = \frac{\Delta X_{tm,t} \times D_{avg} \times b_{avg}}{P_{avg} \times GL} \times C_{cmpl} \tag{5.5}$$

where:

$D(t)$ = time t creep compliance (in kPa);

GL = gauge length (0.038 meters for 150 mm diameter samples);

D_{avg} = average diameter of three samples;

b_{avg} = average thickness of three samples;

P_{avg} = creep load average (kN);

$\Delta X_{tm,t}$ = mean of six normalized sample faces with horizontal deformations at time *t;*

C_{cmpl} = correction factor

$$0.6354 \times (\frac{X}{Y})^{-1} - 0.332 \tag{5.6}$$

where:

$\frac{X}{Y}$ = the ratio of the normalized horizontal trimmed mean as an absolute value

Equation 6 provides a correction factor that can be used to account for both vertical and horizontal correction factors because it is non-dimensional; it also accounts for any horizontal bulging in samples under load. The restrictions in Equation 6 are provided by Equation 7:

$$\left[0.704 - 0.213\left(\frac{b_{ave}}{D_{ave}}\right)\right] \leq C_{cmpl} \leq \left[1.566 - 0.195\left(\frac{b_{ave}}{D_{ave}}\right)\right] \tag{5.7}$$

5.4 Results and discussion

5.4.1 Rutting results

The output of the rutting tests executed on the FRT (60°C) are displayed in Figure 10 (BP and LSP) where the depth of the rutting is plotted as a function showing the cycle numbers in terms of percentage. This is shown, left and right respectively, as a linear and as a logarithmic scale graph.

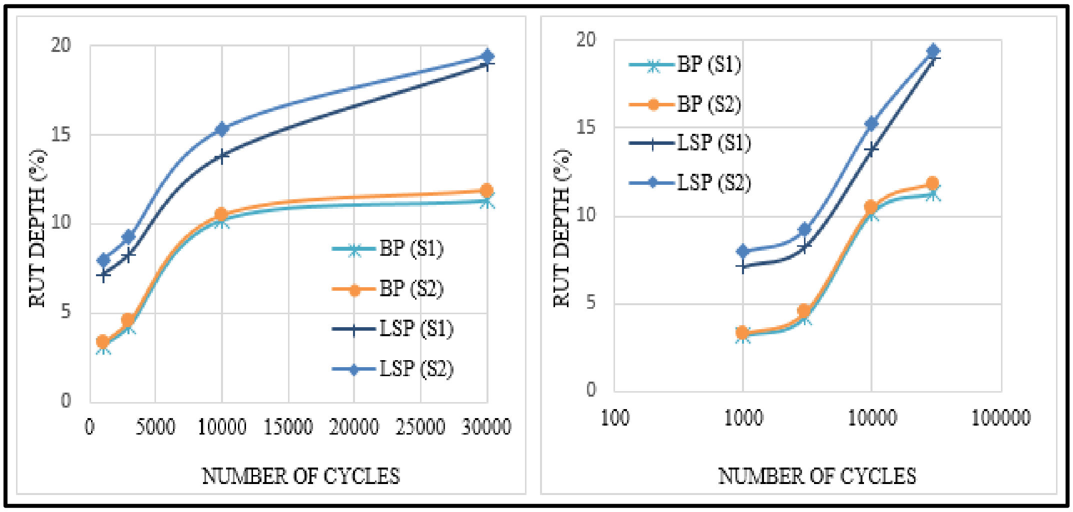


Figure 5. 10 BP and LSP rut test values on liner and logarithmic scales

Figure 11a shows the rut depth mean with the standard deviation for the tests at 60°C. Figure 11b shows the average results between the mean BP mixture compared with the mean LSP mixture. Rutting depths vary from 11.58 to 19.20% at 30,000 cycles. The dashed lines show the BP to be lower (13% max. dif.) and not as scattered (20% max. dif.) as the LSP mixtures (solid lines).

The results shown in Figures 10 and 11 indicate that the BP mixture is generally stable when the rutting levels run from 10,000 to 30,000 cycles, as predicted. Two LSP samples had increased rutting levels from 3,000 to 30,000 cycles; in these samples, the rutting was outside the acceptable parameters. As such, the BP had a sizeable effect on the results. In contrast, rutting is apparent in the LSP mixture.

Table 7 shows that average LSP values have a notably higher degree of rutting compared to BP. An analysis of the standard deviations indicates scattered results in the LSP mixture. Nonetheless, there is a progressive degrease in the BP mixture.

Table 5. 7 BP and LSP results

Mix ID	Number of cycles	1000	3000	10000	30000
BP	Mean rut depth (%)	3.24	4.39	10.34	11.58
	Variance	0.01	0.02	0.02	0.08
	Standard deviation	0.07	0.15	0.14	0.28
LSP	Mean rut depth (%)	7.56	8.76	14.56	19.20
	Variance	0.17	0.24	0.53	0.05
	Standard deviation	0.42	0.49	0.73	0.23

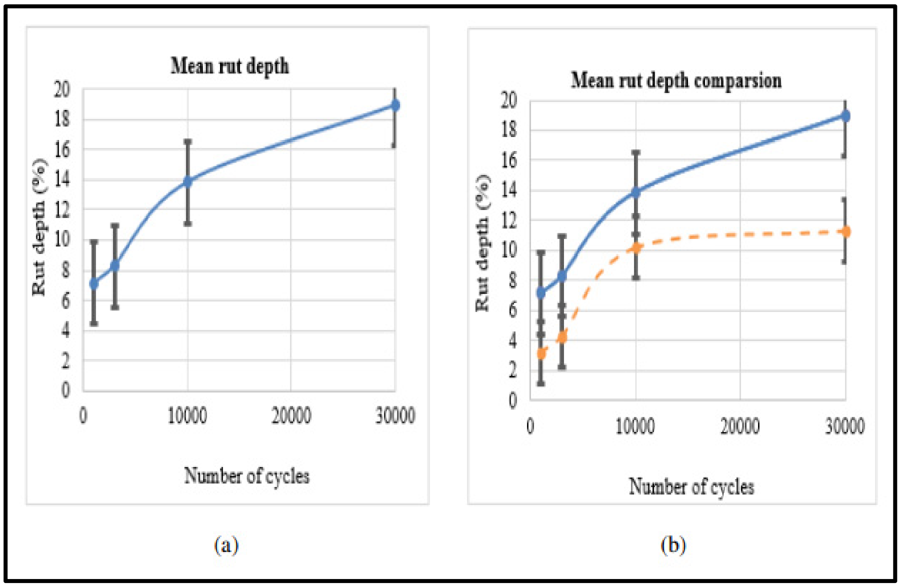


Figure 5. 11a Test values for mean rut depth at 60°C, Figure5. 11b shows the comparison between BP and LSP mixture rutting test results at 60C

5.4.2 ITSM results

Following the ASTM D 4123 standards, the ITSM tests, carried out at 40°C, correspond to the cycle periods of 3, 2 and 1 seconds where the rise time (of 124 milliseconds) and the horizontal deformation (5 µm) stay constant, so that the pulse repetition time is clear.

The ITSM results are shown in Table 8, along with the respective rate of change (Δ) (as a percentage) corresponding to the Stiffness Modulus (SM) at 3, 2 and 1 seconds, where the SM is an average of the 18 samples used in the test.

Table 5. 8 SM at 40°C with a 124 millisecond rise time against cycle period

Mix ID	Stiffness Modulus [MPa]		
	3 s	2 s	1 s
PB	3563	3594	3623
Δ	-	0.87%	1.68%
LSP	2987	3045	3156
Δ	-	1.94%	5.66%

The asphalt pavement samples show rising SM when the cycle repetition period decreases. Nonetheless, the difference between the values are minor, and the maximum variation is 5.66%. The pulse repetition variation period has less effect on the SM than predicted. What is most important in the ITSM is the rise time of the SM. The period of pulse repetition and the rest time are of lower importance.

Table 9 shows the ITSM test results; the rate of change (Δ, as percentage) of the Modulus at 150 milliseconds is compared with the values at 50, 100 and 124 milliseconds. SM values were analysed as an average of 3 samples.

Table 5. 9 Effects of rise time on SM

Mix ID	Stiffness Modulus (MPa)											
	Temperature 5°C				Temperature 25°C				Temperature 40°C			
	50 ms	100 ms	124 ms	150 ms	50 ms	100 ms	124 ms	150 ms	50 ms	100 ms	124 ms	150ms
PB	24561	23336	22755	22178	14425	13194	12003	11562	5122	4469	3563	3067
Δ%	10	5.2	2.6	-	24.7	14	3.8	-	67	45.7	16.2	-
LSP	22132	21235	20798	19861	8841	7936	7652	7233	4125	3457	2987	2786
Δ%	11.3	7	4.7	-	22.2	9.7	5.8	-	48.1	24.1	7.2	-

The ITSM test finds the mixture stiffness results for the various conditions. The load area factor is a ratio of rise time over peak load to the subtended area of the initial load-time of the quarter-wave.

The data from Tables 8 and 9 can be used to show that SM variation at 40°C relates to the pulse repetition cycle reducing from 3 to 1 seconds (with a fixed rise-time of 124 milliseconds); this is generally lower than the values available given a constant level for the pulse repetition of 3 seconds where the rise time is reduced from 150 to 50 milliseconds.

This is to say that the test frequencies have greater effect on the SM when due to a variation in rise time as opposed to when due to variation in pulse repetition cycles. With this, the SM step (related to a diminishment in rise time) is bigger for the higher temperatures. Allowing for variation due to mixture type, the SM values are about 10% for 5°C, and over 45% for 40°C.

Figure 12 shows the mix using BP and has a higher SM for 5, 25 and 40˚C than for alternatives. Given a greater SM for mixes using BP at 40˚C suggests that BP improves the SM of the asphalt binder, thereby possibly improving the HMA's rutting resistance.

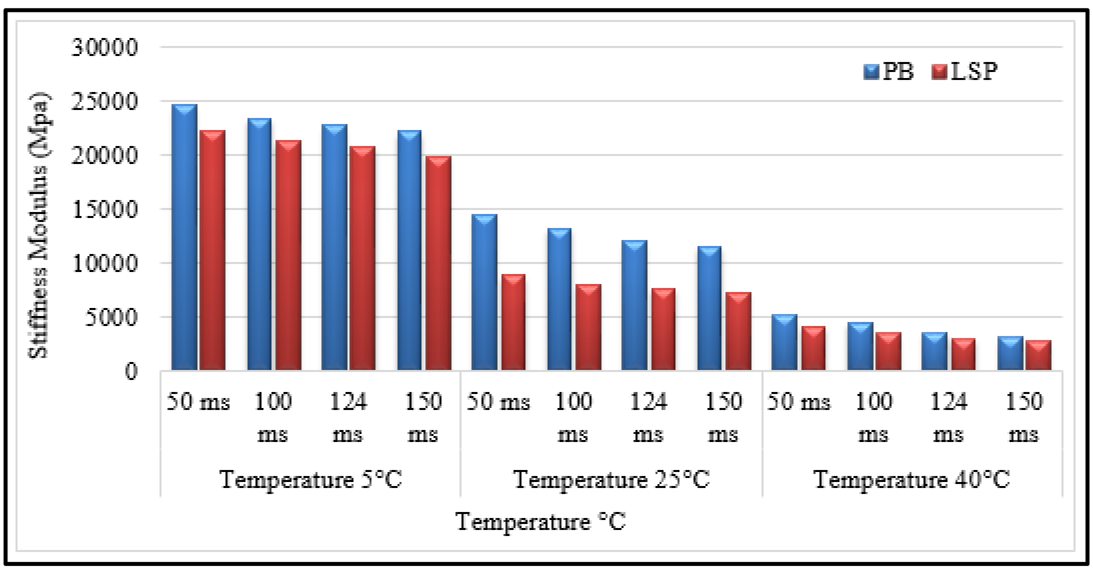


Figure 5. 12 SM for HMA using BP or LSP

ITSM tests show that BP samples have a greater SM when compared with controls. Although LSP has been shown to be better than some alternatives, it is evident from these results that BP is the superior choice overall.

5.4.3 Moisture susceptibility results

Figure 13 shows the ITS of HMA pre and post a freeze-thaw period. In both mixtures, the tensile strength is reduced when there are more freeze-thaw periods. HMA with BP has a higher ITS compared to LSP. Figure 14 shows the TSR of these HMA mixes after each of one, two and three freeze-thaw periods. The HMA with BP has higher TSR than LSP.

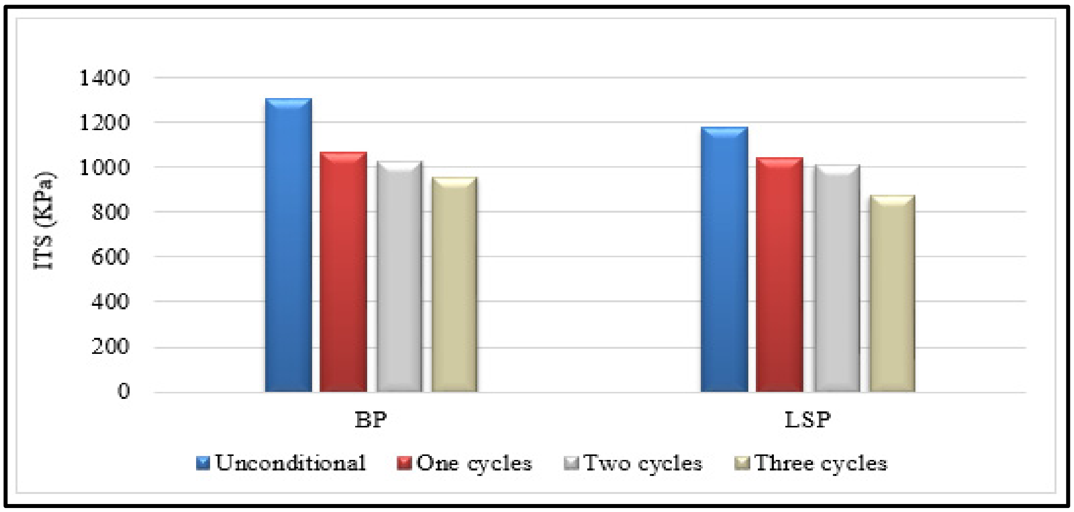


Figure 5. 13 ITS of various HMA mixes

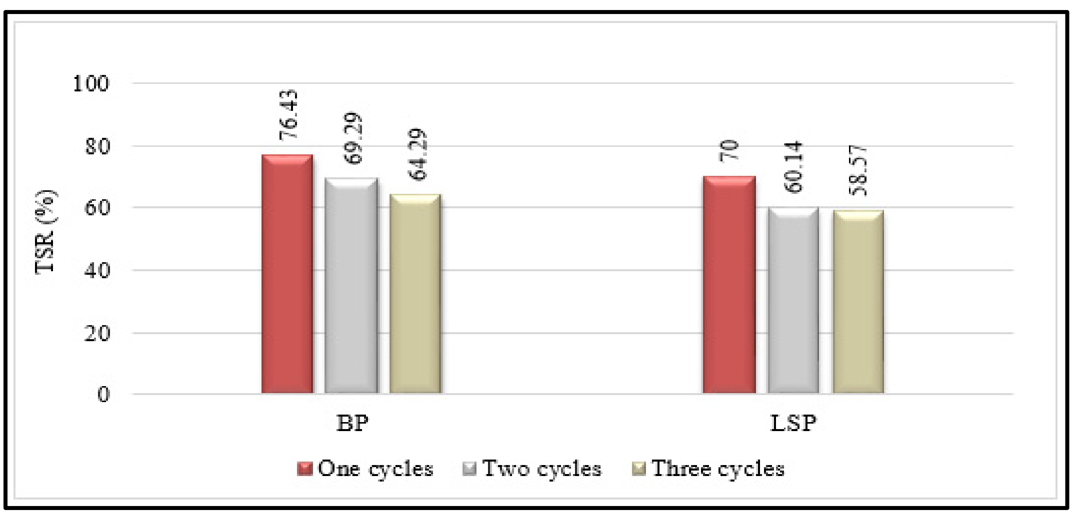


Figure 5. 14 TSR values for various HMA mixes

These data indicate that BP results in greater adhesion among the particles in the aggregate and in the resulting asphalt when compared with the data for LSP. Therefore, using BP gives the mix a greater cohesion and this is evident from the ITS data following each of one, two and three freeze-thaw periods. Beyond that, BP use gives the HMA greater resistance to the aging effects caused by water and other kinds of moisture; this is evident at high TRS values when BP is used, and differs from the lower value data on LSP. This characteristic of BP is due to its basic pH compared to the acidic LSP. LSP's acidity creates detrimental effects in the TRS results, because of its hydrophilic character. The grains of BP have a higher amount of surface contact area and this gives a better ITS test result. The LSP mix is less effective than BP when the test temperature changes. This is clear from the ITS data.

5.4.4 Creep compliance test

The test output data for the LSP mixture and the BP mixture show that, around the core of the sample, the greatest strain response was not more than 100-microstrains or less in most cases. The tests at 40°C with loading times greater than 500 seconds attained 200 microstrains. The three samples were made from each of the two mixtures (i.e. six in total) and then tested at the selected temperatures. Finally, the average of each of these were analysed and discussed. The tests had a constant of a 7% air void at the three temperatures. The stiffest mixtures were made with BP; LSP mixtures were less stiff and had higher creep. As such, the effect that BP has on creep compliance in HMA is significant. The comparative values for creep compliance are shown in Figures 15, 16 and 17.

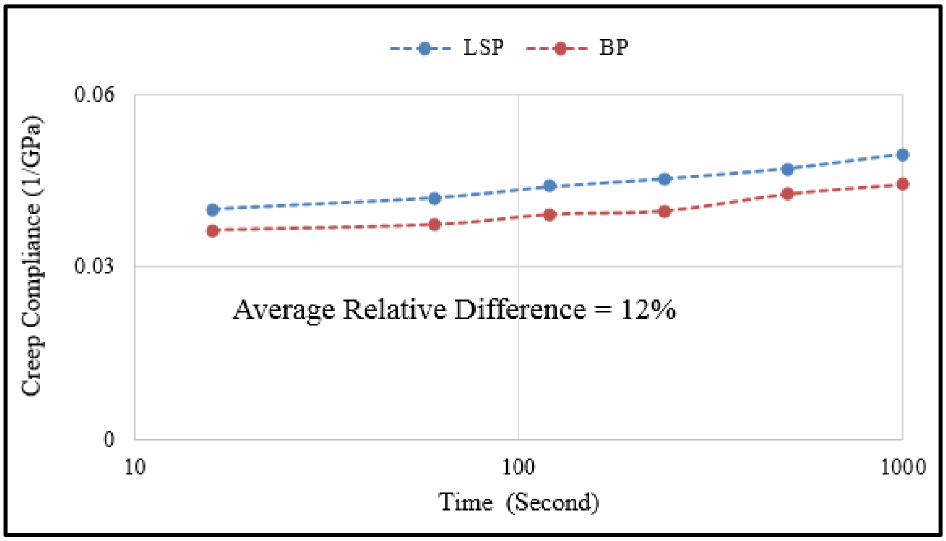


Figure 5. 15 Creep Compliance Comparisons, 0°C

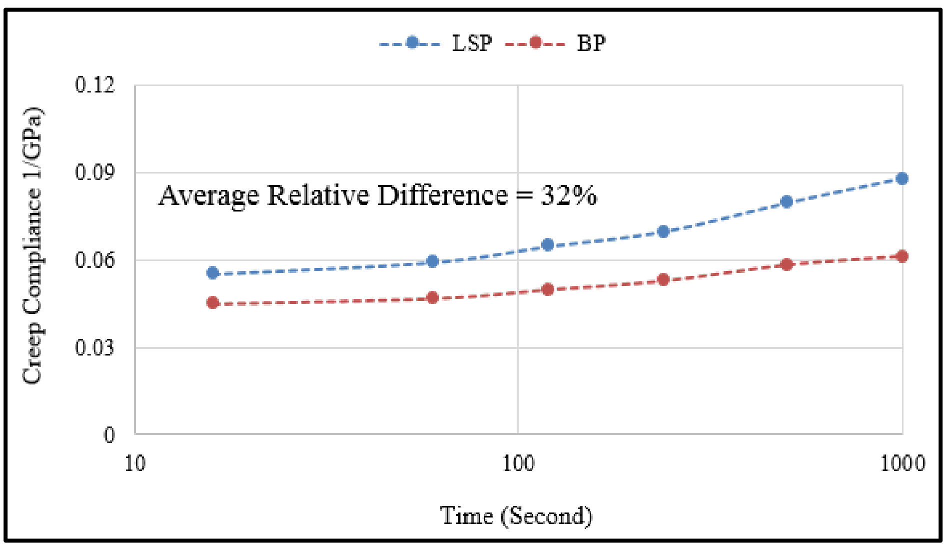


Figure 5. 16 Creep Compliance Comparisons, 20°C

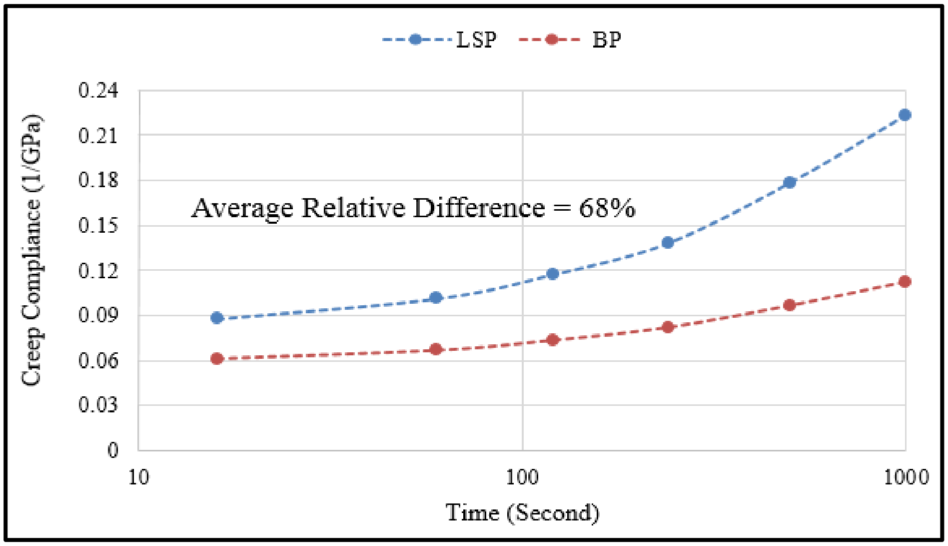


Figure 5. 17 Creep Compliance Comparisons, 40°C

The test results illustrate the different effects of the two fillers on the HMA samples when conducting the 1000 second creep compliance tests. Figures 15,16 and 17 show the effect of temperature on the two fillers, for all of the six indicated loading times of 16, 60, 120, 240, 500 and 1000 seconds. From the graphs, it is clear that the creep compliance curves are markedly different. Temperature for the tests had the greatest effect on the result and time was a secondary but still significant factor. In both mixes, the only variable was the filler. The overall trend was that LSP showed more creep than BP and this trend became significantly more marked at higher temperatures. Even at the lowest test temperature of 0°C, the average relative difference between the two samples was 12%, where BP showed less creep than LSP. At the intermediate test temperature of 20°C, the average relative difference between the samples was 32%, again favouring BP, especially as load times became greater. At the highest test temperature of 40°C, the average relative difference between the samples was a very marked 68%, greatly favouring BP over LSP. The gap between BP and LSP sample test performance grew noticeably after the 60 second point in the two higher temperature tests and thus could be considered an approximate point of divergence between the two curves. After this point, the LSP showed significantly greater creep, especially at higher temperatures. These results clearly show that BP is an important mix filler and should be considered for HMA road construction in hot and arid regions.

5.5 Conclusion

The main objective of the research was to evaluate waste BP as a possible mineral filler in HMA. A number of laboratory tests were conducted to analyse the properties of HMA by testing HMA made with either waste BP or LSP as filler. The test results can be understood as follows:

- As shown by chemical analyses, the BP contains notable amounts of both alumina and silica. As such, it can be considered an aluminosilicate.
- Using BP instead of LSP in HMA makes the mix easier to work with and gives it greater final stability.
- At 40˚C, the HMA made with BP showed a greater ITSM, which allows for more resistance to rutting than the LSP mix.
- ITSM test results show that the greatest SM can be obtained from HMA samples with BP. The SM in the LSP specimens differs from the greatest SM by under 10%.
- BP filler in an HMA may heighten the fatigue life and the water sensitivity of the HMA, when compared with LSP filler.
- BP filler may also limit permanent deformation through creep tests at 40˚C, which allows for better resistance to rutting in HMA in high-temperature climates.
- Considering the technical, budgetary, and environmental perspectives, the adoption of BP as a filler in HMA is recommended over LSP.
- Following the promising laboratory tests, these findings should be replicated in field testing.

5.6 Acknowledgements

The first author gratefully acknowledge technical support from the XRF laboratory staff at the Libyan Petroleum Institute as well as the Pavements and Bituminous Materials Laboratory (LCMB) at the École de Technologie Supérieure (ETS) of the University of Quebec.

DISCUSSION AND CONCLUSION

DISCUSSION

Over the course of this research, PG70-10 bitumen and locally available materials, including natural desert sand and brick powder, were used to improve performance and to reduce rutting deformation of ACP. Several factors must be engineered to attain the desirable results. Several key factors are important here, namely how easy it is to process and to mix these materials. The ratio of natural to crushed sand, the type of bitumen, and the use of filler can significantly affect the stability and the pavement performance. To achieve our target, two bitumen binders (PG70-10 and B60/70), two types of fine aggregate (crushed sand and natural sand) and two types of fillers (lime stone powder and brick powder) were used. Several preparation methods and tests were used to obtain better performance with the desired properties.

CONCLUSION

This research is based on analyses of laboratory experiments on the rheological behaviour of sand-asphalt mixes for HMA design; these mixes were tested to make sure that they met design criteria, including stability and rutting deformation. The materials used, the scope and all the detailed experimental programs and analyses are found in three different papers, all of which are presented in this manuscript-based Ph.D. thesis.

This study focuses on the mechanical behaviour of natural desert sand and how it is affected by both the bitumen choice and the selection of mineral filler. This research contributes to the understanding of sand-asphalt materials, particularly how each of these aspect (i.e. fine aggregate, binder, and mineral filler) affects the HMA's mechanical properties. Results have shown that the effect of using the local available materials are important to consider during the design life of ACPs. The overall findings are listed below.

- The compaction and curing temperatures affect the behaviour of sand-asphalt; by comparing the results of the hardened bitumen, it is also possible to choose between the two binder types. The results indicate that the compaction level significantly affects the behaviour of the mix after a large number of loading cycles.

- Samples with high percentages of voids have a much higher latent fusion heat than the samples with low void percentages. These show the importance of the compaction effort which results in different densities.
- The rutting behaviour of B60/70 samples change with a higher rate of wheel cycles at high temperatures.
- The initial stiffness modulus of the B60/70 mix was much lower than for PG70-10 in the rutting deformation tests. The results of the complex modulus test showed stiffness reduction and a change in viscoelastic behaviour at high temperature cycles. From this, it is concluded that the degradation of the B60/70 mix started rutting during 3000 cycles.
- In hot and arid environments, the rutting test results indicate that the PG70-10 mixture has much better performance with respect to the resistance to rutting deformation than B60/70 mixture.
- The slope of the creep curve is steeper for the B60/70 mix than the PG70-10 mix which results in lower rutting life at high strain amplitude (heavy truckloads) for the PG70-10 mix.
- Results of this study indicate that replacing limestone with brick powder improves resistance of the mixes to rutting and cracking. Results also show that using brick powder decreases the deformation and creep compliance in ITSM tests.
- Comparison of the brick powder mix and the limestone mix show that the limestone mix results in more creep than the brick powder mix and that this trend became significantly more marked, especially at medium and high temperatures;
- The improvement effects of Performance Grade binder and different fillers were assessed to compare the stability and the rutting deformation behaviour after 30,000 cycles of wheel passes. The results demonstrated that the ability of PG70-10 as a binder can increase the performance of the asphalt mix when subjected to high temperature rutting and heavy loads. The rutting results also show that the brick powder increases the durability of the mix after wheel cycle passes;

- In general, PG70-10 mixes are less problematic at high temperatures. This is due to the stability of the mix and the respective cohesion development kinetics.

RECOMMENDATIONS

In this thesis, analyses of different laboratory experiments were conducted to achieve an understanding of the mechanical behaviours of sand-asphalt mixes. Based on the observations from this study, the following recommendations are suggested:

- More work is needed to develop criteria for more loading tests that allow for different frequencies, temperatures, and pavement conditions;
- Tests on the effect of high temperatures on HMA could be improved by performing rutting analyzer tests. In addition to this, results should consider the number of wheel passes on the road surface in hot conditions;
- The outcomes of laboratory experiments show promise concerning production processes for sand-asphalt materials;
- It is recommended that future studies include a life cycle cost analysis to help choose the optimum HMA structure and material;
- The durability and performance of ACP subjected to repeated daily hot wind currents has not been studied yet. There have not been any rutting criteria to take into account daily repeated high temperature winds.

APPENDIX I

PERFORMANCE TESTING OF PAVING MIXES FOR LIBYA'S HOT AND ARID CONDITIONS, USING MARSHALL STABILITY AND SUPERPAVE GYRATORY COMPACTOR METHODS

Fathi S. Almadwi[(1)] and Gabriel J. Assaf[(2)]

[1,2] École de technologie supérieure (Université du Québec), 1100 Notre-Dame St W, Montreal, QC H3C 1K3 Canada

Conference paper published in International congress and exhibition sustainable civil infrastructures: innovative infrastructure geo-technology, Springer, Cham (2017, July)

Abstract

Asphalt Concrete Pavements (ACP) in Libya's southern desert regions suffer two major challenges: the hot and arid climate, with road surface temperatures reaching 65-70°C, and air humidity below 50%. As such, ACP in Libya develops excessive deformation. The lack of modern testing methods and in-situ monitoring during the use-phase make predicting the performance of new mix designs difficult. This paper aims to provide comparative performance data on paving mixes of different Libya-sourced bitumen grades under simulated climate conditions, and to compare the usefulness of empirical testing methods (Marshall) to Performance Graded methods. Two mixes, one using Bitumen 60/70 (B 60/70) and the other using PG70-10, are assessed with Marshall Stability and Super Gyratory Compactor tests, using modern equipment at the laboratory of the ETS (École de technologie supérieure) faculty of engineering of the University of Québec, Canada. The performance of PG70-10 mixes is found to be superior to that of the performance of the mixes using B 60/70. The PG70-10 mix performed within the requirements of lower-volume roads (≤300 vehicles per day). Also, the results of the Super Gyratory Compactor tests are found to be a better indicator of in-service performance than those given by Marshall Stability tests. These results provide a foundation

for performance-testing of different paving mixes that varied in sand and filler content; these are available for applications in similar arid climates, and may provide significant savings by allowing engineers to substitute local materials, such as the abundant rounded sand in southern Libya, for more scarce and costly materials, such as manufactured aggregate.

1. INTRODUCTION

Libya's roads suffer from excessive deformation (primarily rutting, as well as cracking) under the hot and arid conditions found across the country, especially in the southern desert regions. The asphalt cement binder currently used in roads in the hottest regions is Bitumen 60/70 (B 60/70). The bitumen is produced in local refineries but it underperforms in extreme heat and results in rutting of ACP. In the southern deserts, daytime air temperatures can reach 55°C, resulting in road surface temperatures exceeding 70°C. The problem of selecting proper materials for roads becomes more complicated when older empirical grading tests such as the Marshall Stability Test are used. Such tests assess material qualities under standard conditions but they do not give results in engineering units that would allow the prediction of in-service performance (cracking and rutting) across a range of conditions. This paper discusses two points. First, it selects a grade of binder that can perform under conditions simulating those found in hot arid regions like southern Libya; second, it compares the results of PG and Marshall tests to facilitate a transition to PG-based selection of materials.

2. Background

2.1 Empirical vs Performance Graded Classification

The Strategic Highway Research Program (SHRP) conducted research from 1987 to 1993 to improve the design and testing of asphalt materials, resulting in the SUPERPAVE program. Part of this program is the development of Performance Graded (PG) standards for materials. These standards are based on performance across a range of in-service conditions. Since 1999, PG standards for asphalt have been accepted by the American

Association of State Highway and Transportation Officials (AASHTO) and the American Society for Testing and Materials (ASTM). This is superior to empirical testing which only measures properties under a standard temperature or other fixed conditions. For example, the Bowen Penetration Machine invented in 1888 measured penetrability at 25°C; ASTM and ASHTO grading procedures were developed in the 1960s and included tests to measure the viscosity at 60°C, which better approximates the maximum temperature of a pavement surface (Salem et al., 2014).

Performance Grading specifies a range of temperatures within which a material can be expected to perform to the needs of the project. Specifically, they give average, high, and absolute minimum temperatures. For example, the PG70-10 test indicates when a material undergoes the 7-day average road surface temperature, where it does not exceed 70°C or the absolute minimum temperature does not go below -10C. Therefore, engineers can use PG classification to easily identify which materials are suitable for given service conditions. With these tests, we can select materials that are proven to perform under in-service conditions. Engineers can make these choices before construction and during in-place monitoring of roads. By using the PG grading as a guide, engineers can prevent costly and disruptive failures and reduce the need for premature repairs. When needed, the engineer can select higher grades to account for extra strain on roads, such as temperature spikes outside the normal range, or especially high traffic loads. Standard practice is to go up one PG grade (6°C increase, e.g. PG64 to PG70) from fast to slow moving traffic, and up two grades from fast to stationary traffic, such as intersections and bus stops (Arora, M. G., & Kennedy, 1997).

Where no data exists, the transition to SUPERPAVE methods is difficult to compare; this is especially true with regards to past methods and future construction using PG material specifications. It is also a problem where extreme conditions are outside the parameters to which existing PG graded construction has been specified. This is also complicated by the lack of access to modern testing equipment for determining PG grades. To improve road-building in countries like Libya, it is useful to generate data on materials in extreme heat and aridity. Likewise, it is important to find low-cost, accessible testing that allows results to be compared with existing SUPERPAVE data.

2.2 Transition from Empirical to SUPERPAVE Methods

Studies have already been done to establish temperature ranges and to determine the required PG grades in countries with conditions similar to those found in Libya.

Al-Abdul Wahhab et al. conducted a study of asphalt pavement rutting in Saudi Arabia where they identified key factors in surface deformation and made recommendations for traffic control and material selection. This study did not use SUPERPAVE grading in its material recommendations (Wahhab, Fatani, Noureldin, & Bubshait, 1995). Arora & Kennedy studied conditions and road wear in Saudi Arabia and recommended the use of binder with PG70 or PG76 instead of the PEN60/70 binder in use at the time (Arora, M. G., & Kennedy, 1997).

Asi conducted a study to evaluate mix design methods for use in Jordan. This study created a temperature map for Jordan and conducted a series of tests on samples prepared according to Marshall and SUPERPAVE designs. In particular, they used the following tests: Marshall Stability, Loss of Stability, Indirect Tensile Strength, Loss of Indirect Tensile Strength, Resilient Modulus, Fatigue Life, Rutting Behaviour, and Creep Performance. The map showed that most of Jordan was suitable for PG64-10 and that the southwest, northwest and northeast corners of the country were suitable for PG70-10. This study found that SUPERPAVE mix designs, which prescribed a lower bitumen content than Marshall designs, performed better than the Marshall mixes (Asi, 2007).

Alani, Albayati & Abbas addressed the transition to PG grading for road construction in Iraq. They created a temperature zoning map for that country and recommended grades from PG64-16 (equivalent to PEN 60/70 already in use) to PG76-4 for use in the hottest region (Basra) (Hamed M. H. Alani, 2010).

Salem, Uzelac & Matic have done work to map the road surface temperatures in Libya. They used air temperature data from eight locations across the country to estimate the range of road surface temperatures from 2000-2009 and 2012-2013. Then they applied Long Term Pavement Performances (LTPP) equations for high temperature modelling of the data; these were applied at 50% and 98% reliability. With this work, Salem et al. established 7-day average maximum and absolute minimum temperatures for each location. They used the results of the SHRP model, at a 50% reliability level, to select recommended PG grading. The predictions of the SHRP equations were more severe and so provided a greater buffer against stresses of extreme heat and uncontrolled axle loading. Such conditions are often found on roadways in countries like Libya. Uncontrolled axle loading is a critical factor also cited by studies in other desert countries (Wahhab et al., 1995), (Ahmed, A., & Othman, 2000). As shown in Figure 1, for the central, eastern and most of the western region of the country (88% of the total area), the recommended grade of asphalt binder is PG76-10; for the northwest corner, the recommended grade is PG70-10; for the southwest corner, the recommendation is PG82-10 (Salem et al., 2014).

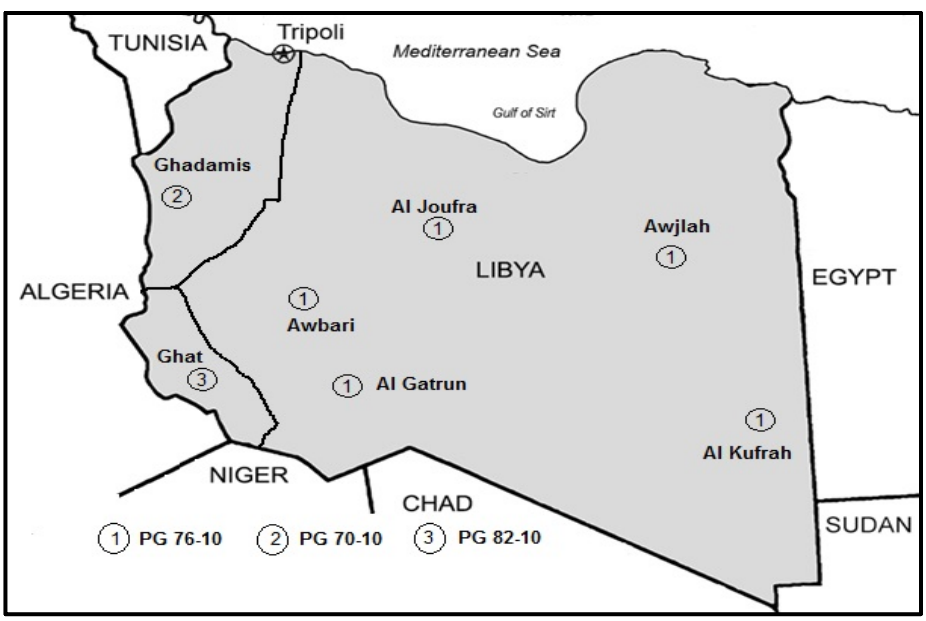


Figure AI. 1 PG zone map of Libya Taken from (Salem et al., 2014)

As this short review demonstrates, the transition to PG grading in road construction is already underway in hot and arid regions such as in north Africa and the Middle East. It is justified because older mix designs have underperformed. There is a need for data to determine which materials may be suitable to the performance parameters identified in these regions. This is important because lower-quality material might pass the Marshall design requirements but might not pass the SUPERPAVE mix design requirements.

3. TESTING METHODOLOGY

For this paper, samples were prepared using a mix of 63.333% crushed sand, 33.167% natural sand and 3.50% filler. Half of the samples used B 60/70 binder and the other half used PG70-10 binder. Three tests were performed on the samples: Marshall Stability, SUPERPAVE Gyratory Compactor, and Rutting Analyzer. All these tests provide useful information about the deformation behaviour of the asphalt in-service. They did this by examining volumetric properties in the case of Marshall and Gyratory Compactor tests, and by directly simulating loaded tire passage in the case of the Rutting Analyzer.

3.1 Materials used for Hot Mix Design

3.1.1 Aggregate

The study used aggregates imported from the southern desert region of Libya and standardized manufactured aggregates for the tests and analyses at ETS in Montreal, Canada. The natural sand was 0-5mm and the manufactured sand was the same size. The specific gravity, the bulk specific gravity and the water absorption of the two kinds of aggregate are shown in Table 1. Figure 2 shows the grading of natural Libyan sand and manufactured sand.

Table AI. 1 Aggregate Properties

Aggregate	Apparent Specific Gravity	Bulk Specific Gravity	Absorption (%)
Natural Sand	2.63	2.42	0.33
Manufactured Sand	2.639	2.44	0.58

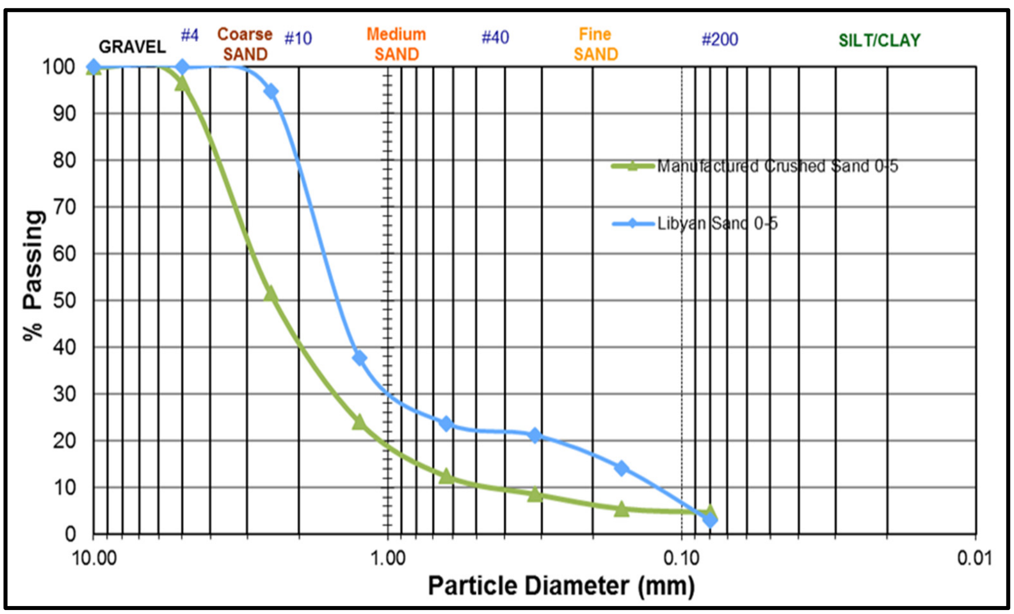


Figure AI. 2 Grading of natural sand and manufactured crushed sand

3.1.2 Asphalt binder

The two asphalt binders used in the study were PG 70-10 and B 60/70. The PG 70-10 mixing temperature was recommended to be 162°C and the compaction temperature was recommended to be 154°C. The B 60/70 was recommended to be mixed at 156°C and compacted at 143°C. All recommendations were followed exactly. Figure 3 shows the

Brookfield viscosity as compared to temperature equivalents and how they relate to the binders used here.

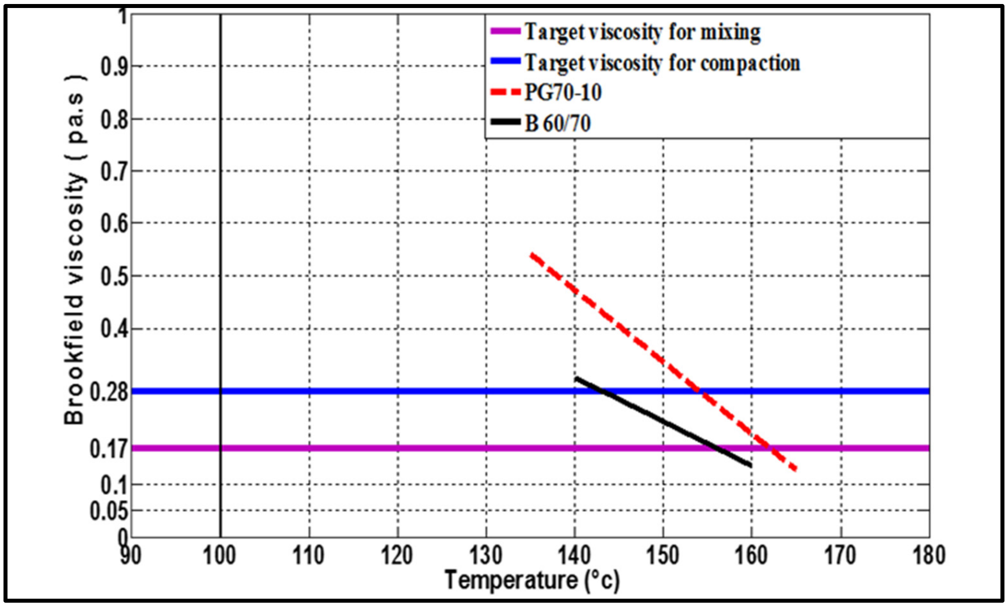


Figure AI. 3 Brookfield temperature vs viscosity curve for asphalt binders

3.2 Hot Mix Asphalt Design Tests

3.2.1 Marshall Stability Test

This test is used to determine the following volumetric properties of the sample: the Optimum Bitumen Content (OBC) of the mix; the stability and plastic flow; the percentage of air voids (Va); the percentage of voids in mineral aggregate (VMA); the percentage of voids filled with asphalt (VFA). In order to determine the OBC, tests were performed with a 5% content; these were insufficient to bind the mix. Further tests were performed with 6% content; these resulted in bleeding of bitumen. Therefore, a bitumen content of 5.5% by weight was chosen for the samples in these tests. All tests followed procedures for determining OBC in Marshall mixes according to the “Basic Asphalt Materials Mixture Design and Testing Technician Training Manual", October 2012.

3.2.2 SUPERPAVE Gyratory Compaction Test

This test is used to produce compacted specimens; these provide data on the air void (Va) content at each stage of the compaction process. The samples were measured at 10, 80, 100 and 200 gyrations. The samples that contained PG70-10 binder were tested at 154°C; the samples that contained B 60/70 were tested at 143°C. The standards were from the ESG-5 category of hot mix paving materials from the Quebec Ministry of Transport (Ministère des Transports**,** Quebec: MTQ). These standards specify targets of **Va ≥ 11%** for N-ini**,** **Va = 4 – 7 %** for N-des**,** and **Va ≥ 2 %** for N-max (Ministère-des-Transports-de-Quebec, 2016).

3.2.3 Rutting Analyzer Test

This test subjects the samples to a loaded wheel in order to simulate the passage of traffic over time. The machine temperature can be adjusted for binders with different softening points; the weighting and tire pressure of the wheel can be adjusted to simulate different vehicle conditions. The first test consisted of 1000 cycles at 600kPa tire pressure at room temperature; following that, tests were made of 1000, 3000, 10 000 and 30 000 cycles (cumulative) at 65°C. The rutting depth was measured after each stage. The MTQ standard specifies a maximum deformation of 10% at 1000 heated cycles and 15% at 3000 heated cycles (Ministère-des-Transports-de-Quebec, 2016).

4. Results

All three tests showed that, overall, PG70-10 performed better than B 60/70 under the specified conditions, as indicated in the next sections.

4.1 Marshall Stability Results

As shown in Table 2, samples containing PG70-10 were more stable and more consistent in measures of air void content than samples containing B 60/70. Table 2 also shows that samples containing PG70-10 were within the target range specified by the MTQ standard. All samples containing B 60/70 produced results exceeding the 2-4mm target for flow, according to the standards referenced above. Each curve in Figures 4a, 4b, 4c represents one of the two binders, PG70-10 or B 60/70.

Table AI. 2 Marshall Stability Test Results

% of Mixture Design 63.333 Crushed Sand 33.167 Natural Sand 3.50 Filler		**Marshall Volumetric Properties**				
	Sample No	**VMA**	**VFA**	**Va**	**Flow**	**Stability**
PG 70-10	**S1**	14.14	78.3	3.08	4.00	23.654
	S2	14.85	73.90	3.88	3.90	20.831
	S3	14.81	74.12	3.84	3.70	20.800
	S4	14.26	77.50	3.21	3.50	21.812
BITUMEN 60/70	**S1**	15.58	67.77	5.02	5.30	22.623
	S2	16.52	63.19	6.08	6.40	20.285
	S3	16.63	62.69	6.21	4.40	12.820
	S4	15.59	67.71	5.03	4.60	14.442
MTQ Standard		Min.9-max.22	Min.60-max.80	3 - 7%	2- 4 mm	Min. 8KN

From Figure 4a, it can be seen that the Marshall Stability of PG70-10 is higher than B 60/70 and therefore it is preferred.

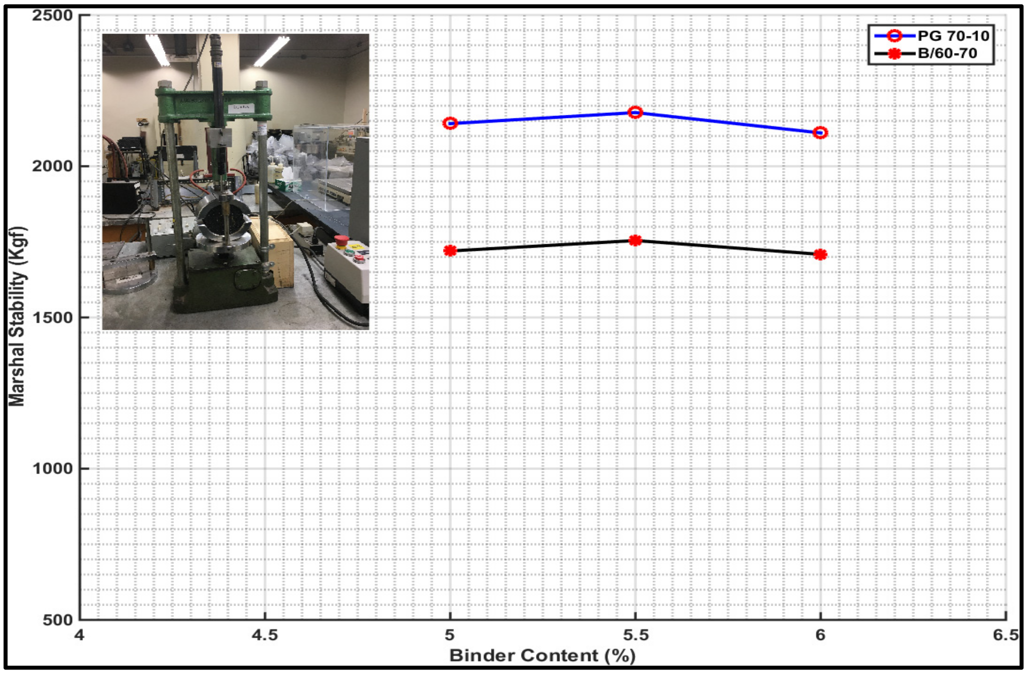


Figure AI. 4a Marshal stability

In Figure 4b, it can be seen that the percentage of Air Voids (Va) in PG70-10 is within the acceptable limit of 4%, whereas the Va% of B 60/70 exceeds this limit.

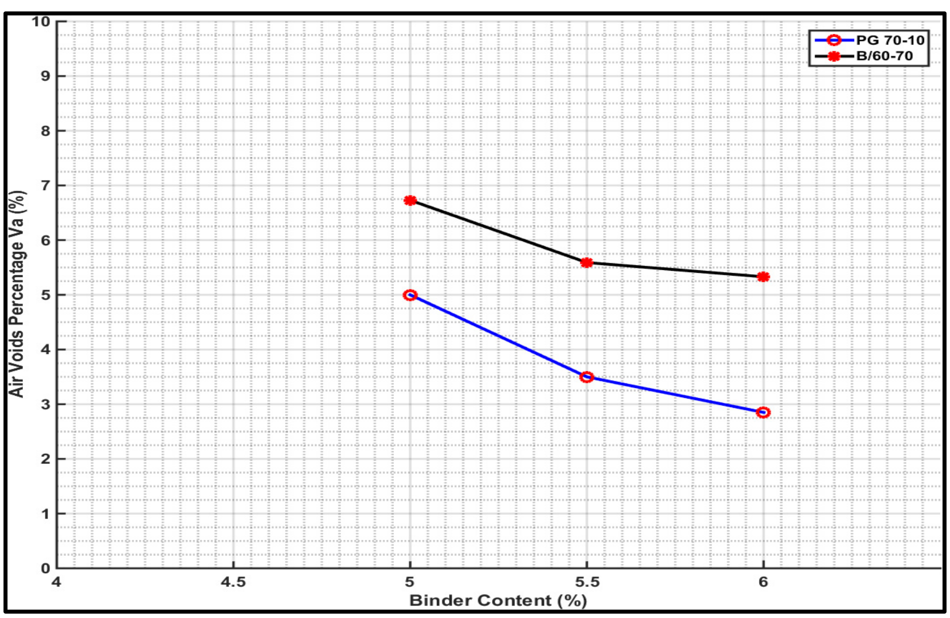


Figure AI. 4b Air Voids percentage Va (%)

In Figure 4c, it can be seen that the Marshall Flow of PG70-10 is between in the ideal target range of 2-4mm, whereas the Marshall Flow of B 60/70 exceeds this limit.

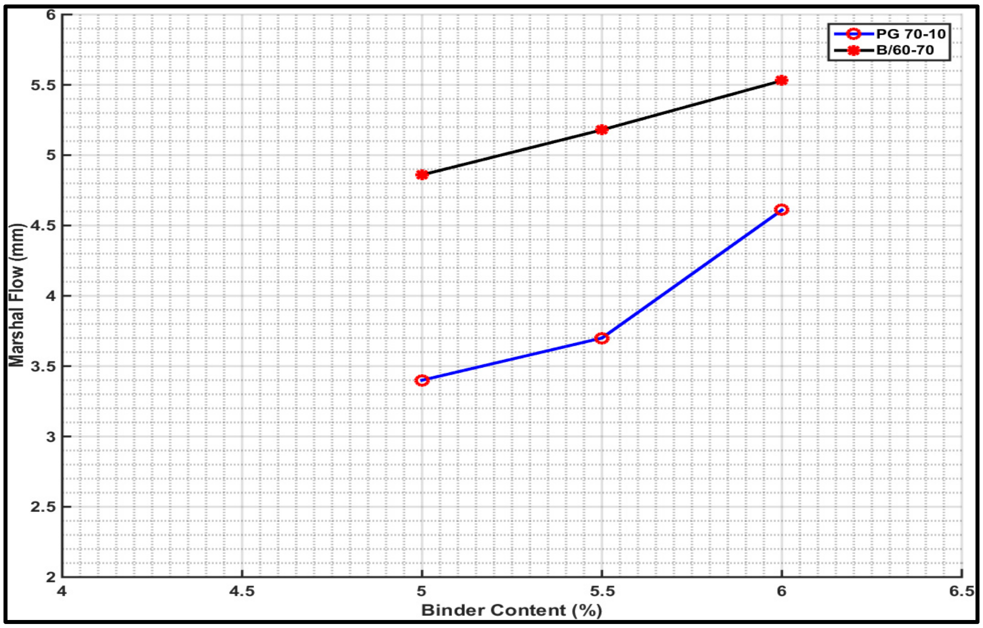


Figure AI. 4c Marshal flow

From these three tests, it is clear that the 5.5% bitumen in the two binders we tested is the best compromise between the desired properties of the hot mix asphalt.

4.2 SUPERPAVE Gyratory Compactor Results

As shown in Table 3, samples that contained PG70-10 had a smaller percentage of air voids than the samples that contained B 60/70. This was true at all gyration counts. Most importantly, the samples containing PG70-10 had air void content within the 4-7% target range for N-des at 80 and 100 gyrations. Both samples containing B 60/70 exceeded that range at 80 gyrations. Figure 5 clearly illustrates this.

Table AI. 3 SUPERPAVE Gyratory Compactor Test Results

SUPERPAVE Gyratory Compacter Mix Design			% Va / gyration			
Binder	**Sample No**	**Compaction Temperature**	**10**	**80**	**100**	**200**
PG 70-10	**S1**	**154°C**	15.25	6.58	5.73	3.5
	S2	**154°C**	14.83	5.78	4.38	3.81
BITUMEN 60/70	**S1**	**143°C**	15.99	7.40	6.65	4.40
	S2	**143°C**	16.77	8.23	7.12	5.34
Specification Tolerance			≥ 11	4 -7	4 -7	≥ 2

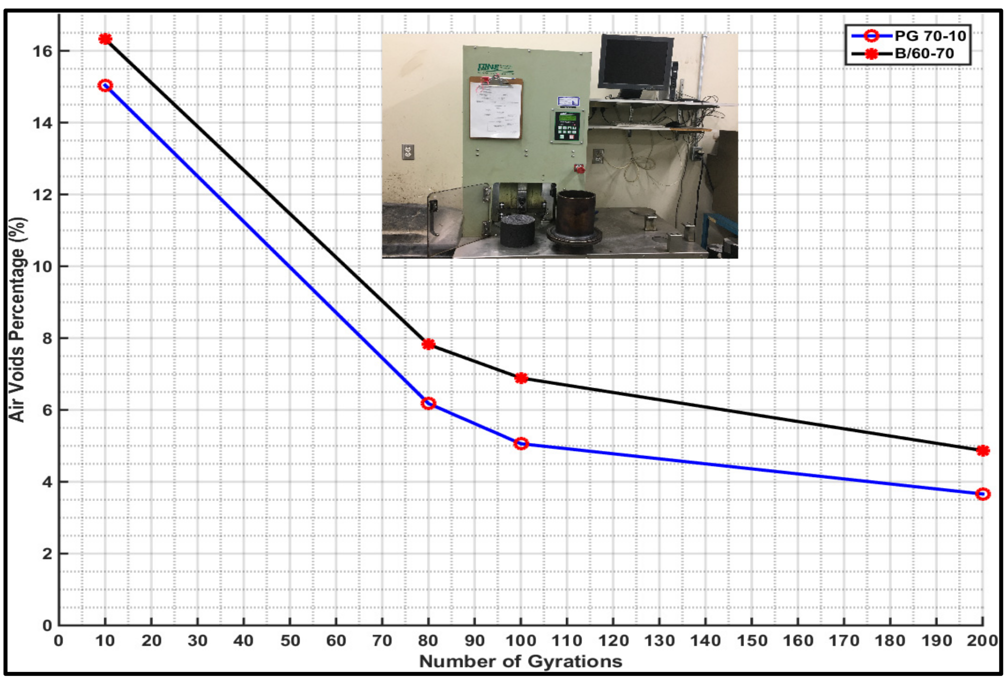


Figure AI. 5 Superpave Gyratory Compactor (SGC) Test

4.3 Rutting Analyzer Results

As shown in Table 4, samples containing PG70-10 showed less rutting than samples containing B 60/70 at all stages of the rutting test (1000-30000 cycles cumulative). At 3000 cycles, the PG70-10 samples lost an average of 2.25% of their original height; in contrast, after 3000 cycles, the B 60/70 samples lost an average of 4.34% of their original height. Figure 6 clearly shows this.

Table AI. 4 Rutting Analyzer Test Results

% of Mixture Design 63.333 Crushed Sand 33.167 Natural Sand 3.50 Filler			Rutting %					Rutting at 3,000 Cycles (mm)
			Cold Cycle	Hot Cycles (Cumulative) at 65°C				
Binder	**Sample No**	**Sample Height**	**1000**	**1000**	**3000**	**10 000**	**30 000**	
PG70-10	**S1**	50.31	1.29	3.17	4.54	10.20	11.30	2.28
	S2	51.21	1.01	3.14	4.31	8.50	9.90	2.21
BITUMEN 60/70	**S1**	51.17	2.91	7.14	8.27	13.83	18.97	4.23
	S2	52.16	3.21	7.52	8.51	13.10	18.70	4.44

Looking at the data from Table 4, as displayed in Figure 6, it can be seen that the slope of PG70-10 changes noticably after 10 000 cyles. From 1000 to 10 000 cyles, the slope of the PG70-10 is comparable to the slope of B 60/70. But from 10 000 to 30 000 the results from the PG70-10 tests have a much lower slope. This is where the slope of PG70-10 contrasts the most with the slope of the B 60/70 results. This has important implications for a road surface in actual use. Where the B 60/70 undergoes considerable ongoing compaction, the PG70-10 shows much more general stability over time.

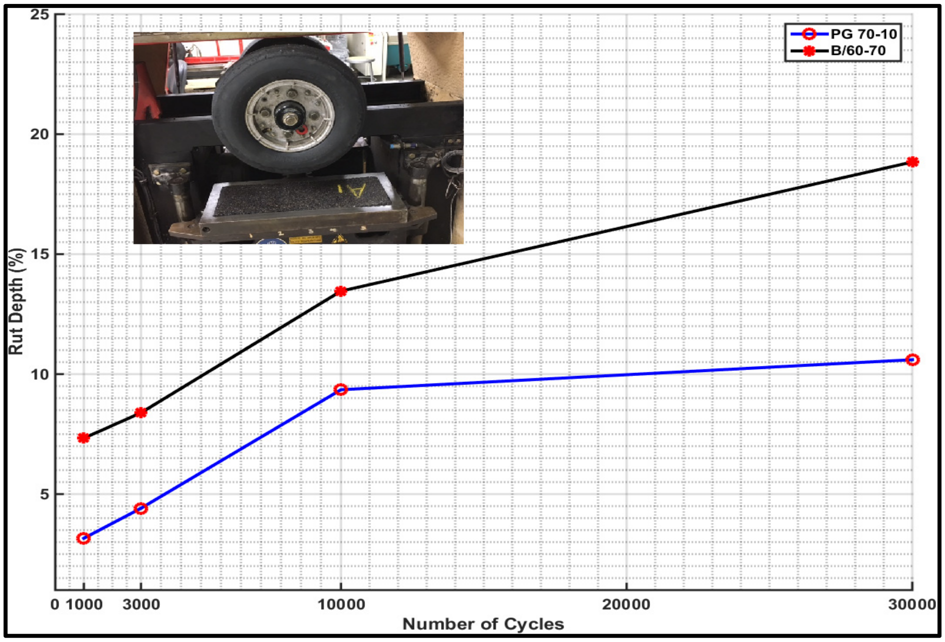


Figure AI. 6 Rutting results

5. CONCLUSIONS AND DISCUSSION

The lab tests demonstrated the superior performance of PG70-10 binder in asphalt mixes subjected to conditions simulating those found in Libya's southern desert regions. This supports the recommendations found in previous literature that examines deformation in rutting of asphalt pavements in countries like Libya. The test results also show that the SUPERPAVE Gyratory Compactor predicts in-service performance more accurately than the Marshall Stability test.

6. FUTURE WORK

Future research will investigate the performance of different additives in the mixes. These additives include polymers or fibres. Other research will include varying the ratio of manufactured to natural sand. The information gained by these studies will provide data resources to engineers planning a new generation of road construction in countries like Libya. This will facilitate the transition to SUPERPAVE-compliant methods. It will also help to construct better-performing roads with lower maintenance costs.

7. ACKNOWLEDGEMENTS

The first author wishes to thank ETS for providing the laboratory equipment for this research.

APPENDIX II

FINDING AN OPTIMAL BITUMEN AND NATURAL SAND BALANCE FOR HOT MIX ASPHALT CONCRETE IN HOT AND ARID REGIONS

Fathi S. Almadwi(1) and Gabriel J. Assaf(2)

1,2 École de technologie supérieure (Université du Québec), 1100 Notre-Dame St W, Montreal, QC H3C 1K3 Canada

Conference paper published in the solving pavement and construction materials problems with innovative and cutting-edge technologies. Geo-china 2018. sustainable Civil Infrastructures. Springer, Cham:

Abstract

Designing a Hot Mix Asphalt Concrete (HMAC) for Libya's southern desert regions has two serious challenges. Due to the extremely hot and arid conditions, where road surfaces can be reaching 70°C, and with air humidity well below 50%, a HMAC in the southern desert often suffers great deformations in the form of rutting. This is particularly a problem in lower-volume roads because the remoteness and the low traffic of these roads makes an investment in their construction and repair not cost effective to use higher-grade materials. This paper shows comparative data for paving mix performance of two bitumen grades available in Libya, both under simulated climatic conditions. These are used to select the best mix of sand and bitumen. The research first determines the best mix of natural desert sand with manufactured aggregate. Then it uses this sand mix as a control to evaluate two kinds of bitumen, Bitumen 60/70 (B60/70) and PG70-10. With a fine aggregate mix of 33% natural desert sand (from the region) and 63% manufactured crush fine aggregate, the performance of PG70-10 was found to be superior compared with B60/70. These are assessed with the Marshall test, the Super Gyratory Compactor test, and the wheel track test in the ETS laboratories (École de technologie supérieure) faculty of engineering of the University of Québec, Canada. This finding is

important for designers working in arid conditions as a way of sizable cost savings by substituting available materials, i.e. the rounded sands of desert regions for more expensive sands imported from other regions.

Keywords: Natural Sand, Hot Mix Asphalt, PG70-10, B60/70, fine aggregate, Libya.

1.1 INTRODUCTION

The roads in the desert areas of southern Libya are constrained by two factors. They must be built on a strict budget because of their low-volume use but they suffer from extreme temperatures in the arid conditions and under the hot desert sun. For this reason, the roads are typified by excessive deformation (mostly rutting and some cracking). These roads are made with Bitumen 60/70 (B60/70) because the bitumen is an established formulation and widely available from local refineries. Due to the extreme conditions, it performs poorly in extreme heat and the result is a large degree of rutting in the Hot Mix Asphalt Concrete (HMAC). At their most extreme, the deserts in the south of the country attain daytime air maximum temperatures of 55°C, which causes road surface temperatures to go over 70°C (Almadwi & Assaf, 2017).

1.2 Scope and Objective of Study

In areas with arid sand deserts, like southern Libya, there are practical reasons to use desert sand in asphalt mix but too much natural sand in HAMC results in an HMAC more likely to show stresses leading to permanent deformation in use. This paper aims to demonstrate the effects of natural sand on the in-service use of asphalt mixture. Here two objectives are sought. The first is to find the optimal balance of natural desert sand with manufactured fine aggregate; the second is to select the better bitumen formulation for the conditions. Once the mix of sand is determined, the two bitumen formulations are mixed, and the different mixes undergo various tests including the Marshall, Gyration, and Wheel Track tests. Results from the tests are used to evaluate the different mixes with the same equipment in order to predict the behaviour of the asphalt mixtures with the materials. Results are analyzed to determine the effects of the different factors in the behaviour of the asphalt.

2. BACKGROUND

2.1 Material Choice and How it Affects HMAC Properties

Every component in the HMAC has an effect on its final characteristics and how these respond to moisture, fatigue leading to cracking and permanent deformation. Other factors, that need to be present in testing in order to properly simulate actual conditions are loading distribution and climate (Epifanio & Gan, 2009).

2.1.1 Aggregate

The quality and consistency of aggregate is an important factor in pavement performance. More specifically, in order to resist permanent pavement deformations, the durability, angularity, and chemical composition of an aggregate must be considered. In certain regions with very high temperatures, concrete mixes can incorporate partially crushed aggregate or aggregates containing natural desert sand. 94-95% of a HMAC is made up, by weight, of aggregates and these aggregates create an effective structural matrix that carries most of the weight of the traffic load. Therefore, aggregate selection is a key stage in designing a quality pavement that will be durable over time (Asphalt Institute, 1998). That is to say, that, when planning a pavement for a long service life (i.e. a pavement that is not prone to permanent deformation) it is the matrix structure of the aggregate that must be kept in mind. The structural matrix that the aggregate creates is designed to receive and dissipate the energy from years of axle loads without itself changing its overall structure. In creating this structural matrix, individual pieces of fine aggregate must be considered for their coefficient of friction (i.e. texture), their form, as well as the gradation of the whole. When these pieces of aggregate have greater coefficients of friction between pieces and have greater angularly in form, the resulting structural matrix will have greater sheer strength than a HMAC made with smooth, and rounded aggregate. Table (1) shown some useful numbers of coefficient of friction on the engineering properties of material. A structural matrix made with aggregate that has a high coefficient of friction and that has more angular, more cubical pieces will be more stable because the aggregate will interlock and work together as an elastic whole; in contrast, a HMAC made from smooth, rounded aggregate will not interlock as much and pieces of aggregate will start to migrate in relation to each other and the matrix will be less stable in the long-term (Perdomo, D., & Button, 1991); (Chowdhury, A., Button, J. W., & Grau, 2001) and (Park and Lee, 2002).

Table AII. 1 Coefficient of Angle Friction (ϕ) on
The Engineering Properties of Material

Angle of Internal Friction (ϕ)	
Rock	30°
Sand	30-40°
Gravel	35°
Silt	34°
Clay	20°
Gravel with some sand	34-48°
Silt	26-35°

2.1.2 Asphalt Binder

The quality of the bitumen, with regards to the aggregate type, will affect the flexibility, stability, fatigue resistance, and durability of the asphalt mix, as well as how it resists moisture damage (Hay, Richard , E., & Peter, 1986). Too much bitumen leads to bleeding under traffic loads and poor pavement performance. The properties of the hot asphalt mix are affected by the physical properties of the bitumen (Roberts, F. L., Kandhal, P. S., Brown, E. R., Lee, D. Y., & Kennedy, 1996). The conventional B60/70 penetration grade bitumen, commonly used in road projects in hot areas. For reference, the hardness of the bitumen is evaluated based on how far a specified needle will vertically penetrate the sample within five seconds at 25 C. Therefore, the softer bitumen is the higher penetration number. The number is shown as a range to the tenth of a millimeter; therefore B60/70 would indicate that the needle penetrated to an average range of between 0.6 and 0.7 millimeters.

The softening point of B60/70 is 49-54°C which is below daytime surface temperatures of pavement in Libya, where they often reach 70°C. Options to solve this problem include either a polymer-modified asphalt or a higher, harder grade of bitumen. This project investigates a higher quality of bitumen (Almadwi & Assaf, 2017).

3. METHODOLOGY

3.1 Materials and Testing Procedures

3.1.1 Materials

3.1.1.1 Aggregate

This study was conducted using dry samples obtained from the sand dune area in the southwestern desert of Libya, around the city of Sabha. Sand from the region was collected and subjected to identification to determine its engineering properties, including sieve analysis, specific gravity and absorption. The size of the manufactured sand in the aggregate mixture was 0-5 mm. The volumetric properties for the natural Libyan sand and the manufactured sand is shown in Table 2. Figure 1 shows the grain size distribution for both. In this research, a mineral filler powder from a Canadian manufacturer was used in small quantities (4%) and combined with asphalt sand mixes; the resulting properties are shown in Table 2.

Table AII. 2 Aggregate Properties

Aggregate	Apparent Specific Gravity	Bulk Specific Gravity	(%) Absorption
Natural Sand 0-5 mm	2.63	2.42	0.33
Manufactured Sand 0-5 mm	2.639	2.44	0.58
Mineral Filler	2.70	-	1

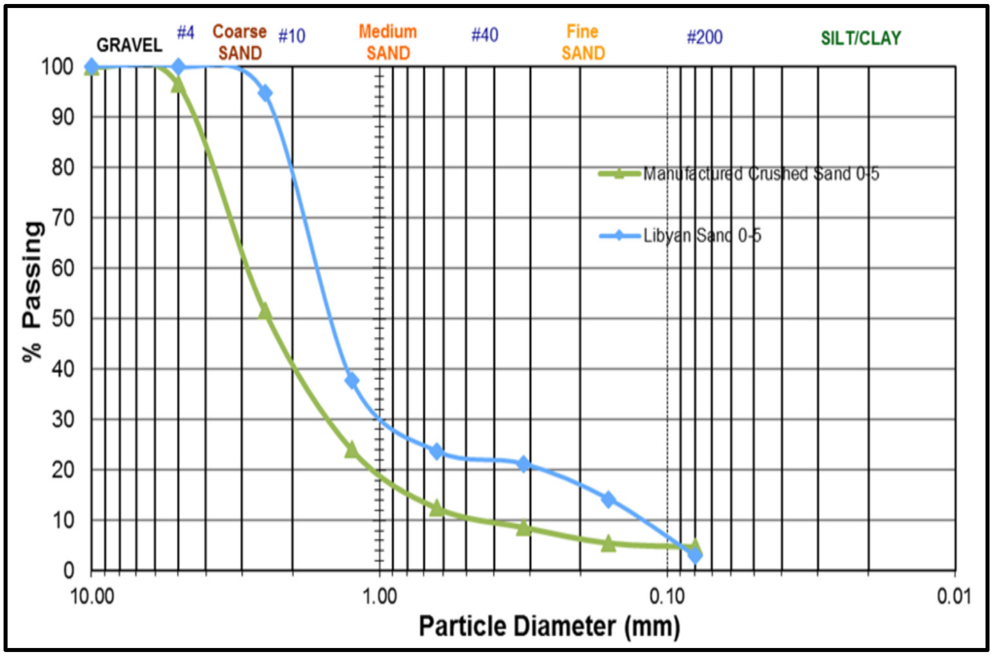


Figure AII. 1 Grain size distribution of fine aggregates

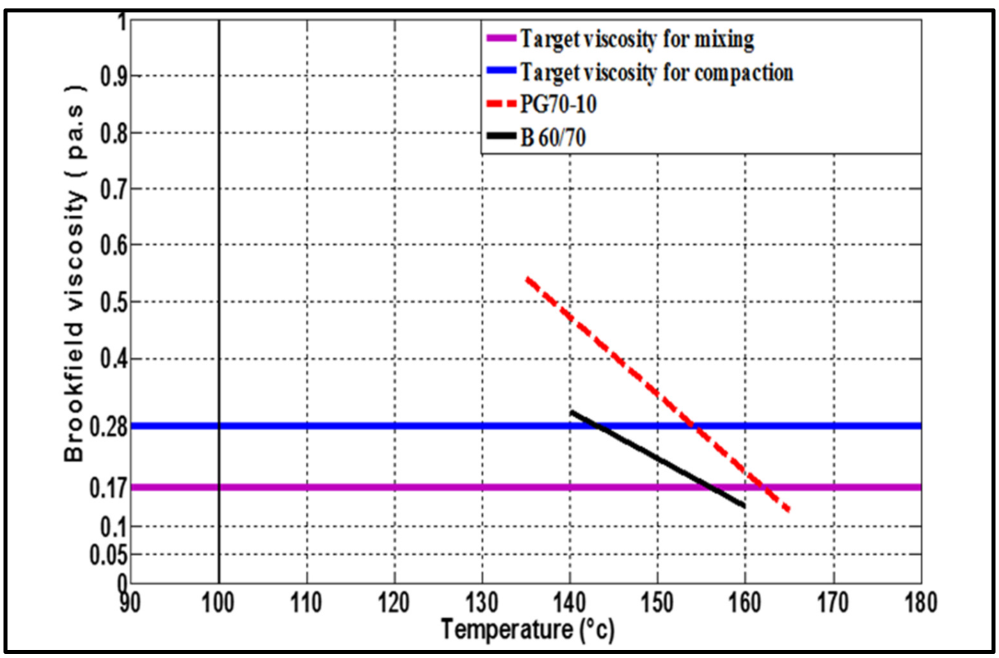


Figure AII. 2 Brookfield temperature vs viscosity curve for asphalt Taken from (Almadwi & Assaf, 2017)

3.1.1.2 Asphalt Binder

Due to the conditions discussed in the background, the solution chosen here is to use bitumen PG70-10 and B60/70. Their recommended temperatures are as follows: PG70-10 is recommended to be mixed at 162°C and compacted at 154°C. B60/70 is recommended to be mixed at 156°C and compacted at 143°C. The recommended temperatures were followed without exception, as illustrated in Table 3. The dynamic shear rheometer (DSR) is used to characterize the viscous and elastic behavior of asphalt binders at medium to high temperatures. This characterization is used in the Superpave PG asphalt binder specification. As with other Superpave binder tests, the actual temperatures anticipated in the area where the asphalt binder will be placed determine the test temperatures used. Figure 2 shows the Brookfield viscosity as compared to temperature equivalents and how they relate to the binders used here.

Table AII. 3 Properties of Asphalt Binders and Temperatures
for Mixing and Compacting

Asphalt Binders	Bulk Specific Gravity	Mixing Temperature	Compacting Temperature
PG70-10	1.29	162°C	154°C
B60/70	1.02	156°C	143°C

3.2 Determining the Optimal Natural Sand Mix

Several types of testing equipment and test procedures were used to determine the amount of natural sand required. For all of these tests, both PG70-10 and B60/70 were used. The tests were carried out with 50% natural sand content, but this mix was found to be insufficient in terms of strength and stability. Following tests were carried out with 25% natural sand content and these showed that the amount of natural sand was not a problem in terms of the final mix quality. Nonetheless, the objectives of the project are to find a workable maximum amount of natural sand; therefore, more tests were done. Following tests used 30% and 35%. At 30%, the resulting pavement showed good characteristics but at 35% the tests showed too much deformation. As a result, a natural sand content of 33% by weight was selected for the samples in this study with regard to the total mass of the aggregate mixture.

3.3 Determining the Optimal Bitumen Content

Optimal bitumen content (OBC) was determined according to the Marshall method. To determine the OBC, the tests were carried out with 4.5% bitumen content, but these were found to be insufficient to bind the mix. Following tests were carried out with 5% bitumen content and these showed that the amount of bitumen was not sufficient. Following tests used 5.5% and 6%. The test with 6% bitumen resulted in too much bitumen bleeding. As a result, bitumen content of 5.5% by weight was selected which corresponded to 4% air voids that were fulfilled the requirements of Marshall method for the samples in the following tests.

3.4 Test Procedure

In Libya, hot asphalt mixtures have been prepared according to the Marshall method, which does not take into consideration the shear strength during the mix design phase. For this reason, it does not select out the mixes that perform poorly with regards to rutting. Present mix design methods, according to (Mahboub, 1991), have gone beyond the scope of what they were meant to do because road use has changed since they were first designed. Examples of differences are that tire pressure, axle-load and environmental conditions are different from what they were even a decade ago. The approach for making the aggregate mix consisted of adding a percentage of natural sand to manufactured sand, 33% and 63%, respectively, by weight of the mix. About 4% of mineral filler was also added.

3.5 Laboratory Tests for HMAC Design

Manufactured aggregate, natural sand, and filler, all between 0 and 5 mm, were used in the laboratory study. The specification of the aggregate gradation for hot mix design is illustrated in Table 4. The natural sand was imported from the desert regions of southern Libya; the bitumen for the tests were PG70-10 and B60/70 grades. Figure 3 shows the specification of aggregate gradations for hot mix design.

Table AII. 4 Fine Aggregate Grain Size Distribution
for Hot Mix Design

Agg Size	**L. Limit**	**U. Limit**	**% Retained**	**% Passing**	**% Cumulative Retained**
14	100	100	0.00	100	0
10	95	100	2,54	97.5	2.54
5	85	100	4.68	95.6	7.22
2.5	50	70	32.25	65.4	39.47
1.25	24	34	39.99	29.6	79.46
0.63	16	24	13.49	17.4	92.95
0.315	14	22	3.96	14.1	96.91
0.16	8	12	1.94	9.8	98.85
0.08	4	8	0.83	5.4	99.68
Pan			0.32		100.00

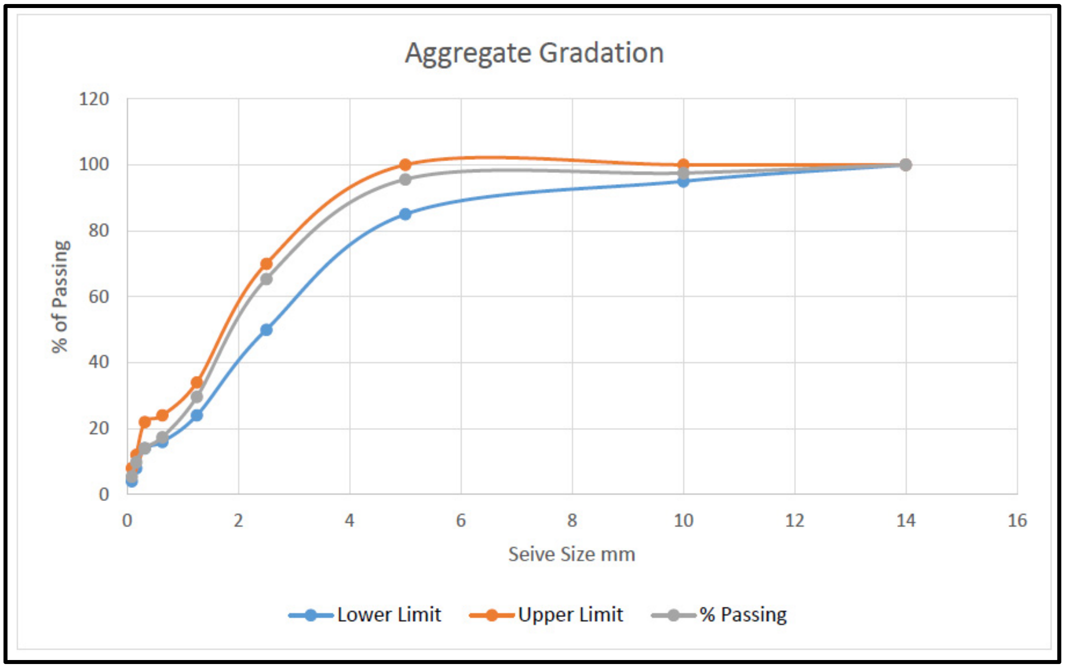


Figure AII. 3 Specification of aggregate gradation for hot mix design

3.5.1 Marshall Stability Test

Test specimens were prepared by applying 50 blows to the top and to the bottom of the samples with a Marshall hammer. Of the samples, half-used PG70-10 binder and the other half-used B60/70 binder. The tests determined the following volumetric properties of the samples: the percentage of voids filled with asphalt (VFA); the stability and plastic flow; the percentage of voids in mineral aggregate (VMA); the percentage of air voids (Va); the Optimum Bitumen Content (OBC) of the mix.

3.5.2 SUPERPAVE Gyratory Compaction Test

The compacted specimens produced by these tests provided data on HMAC at each gyration of the compaction procedure. Samples were measured at the following intervals: 10, 80, 100 and 200 gyrations. PG70-10 binder samples were compacted at 154°C; the B60/70 binder samples were compacted at 143°C.

3.5.3 Wheel Track Test

The test samples were subject to a loaded wheel test to simulate the passage of traffic over time. Binders with different softening points made it necessary to adjust the machine temperature; likewise, tire pressure and the weight of the wheel were adjusted to simulate different axle loads. The first test specified 1000 cycles at 600kPa tire pressure (at standard room temperatures of ~20°C); tests were then carried out at 1000, 3000, 10,000 and 30,000 cycles (cumulatively) at 65°C. After each stage, the rutting depth was measured. Following the standards set by the Ministère des Transports de Quebec (2016), the limit for how much the sample can lose from its original height (in mm), following a certain number of heated cycles (cumulative) at 65°C for the samples is 10% at 1000 heated cycles and 15% at 3000 heated cycles.

4. RESULTS

4.1 Results of the Marshall

The results of the Marshall tests are seen in Figure 4. For the Volume of Mineral Aggregate (VMA), the results are as follows. VMA with PG70-10 there were two samples: the first sample (S1) resulted in 14.14; the second sample (S2) resulted in 14.26. VMA with Bitumen 60/70 there were also two samples: the first sample (S3) resulted in 16.52; the second sample (S4) resulted in 16.63. For the Void Filled Asphalt (VFA), the results were as follows. VFA with PG70-10 there were two samples: the first sample (S1) resulted in 78.3; the second sample (S2) resulted in 77.5. VFA with Bitumen 60/70 there were also two samples: the first sample (S3) resulted in 63.19; the second sample (S4) resulted in 62.69. For the Air Void percentage (Va%), the results were as follows. Va% with PG70-10 there were two samples: the first sample (S1) resulted in 3.08; the second sample (S2) resulted in 3.21. Va% with Bitumen 60/70 there were also two samples: the first sample (S3) resulted in 6.08; the second sample (S4) resulted in 6.21. For the Marshall Flow tests, the results were as follows. Flow with PG70-10 there were two samples: the first sample (S1) resulted in 4; the second sample (S2) resulted in 3.5. Flow with Bitumen 60/70 there were also two samples: the first sample (S3) resulted in 6.4; the second sample (S4) resulted in 4.4. For the Stability test, with PG70-10 there were two samples: the first sample (S1) resulted in 23.654; the second sample (S2) resulted in 21.812. VMA with Bitumen 60/70 there were also two samples: the first sample (S3) resulted in 20.285; the second sample (S4) resulted in 12.82.

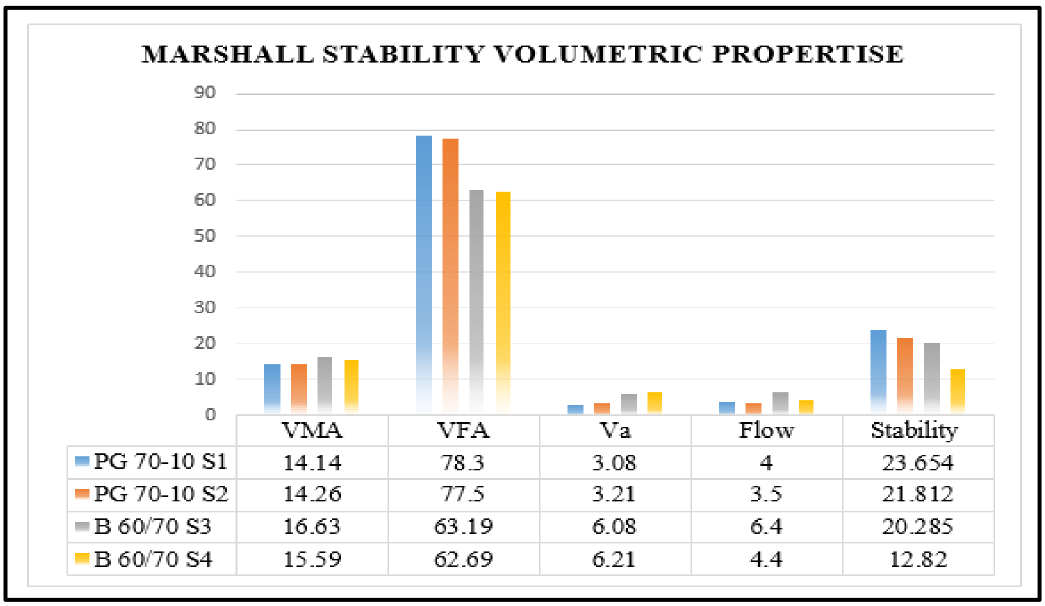
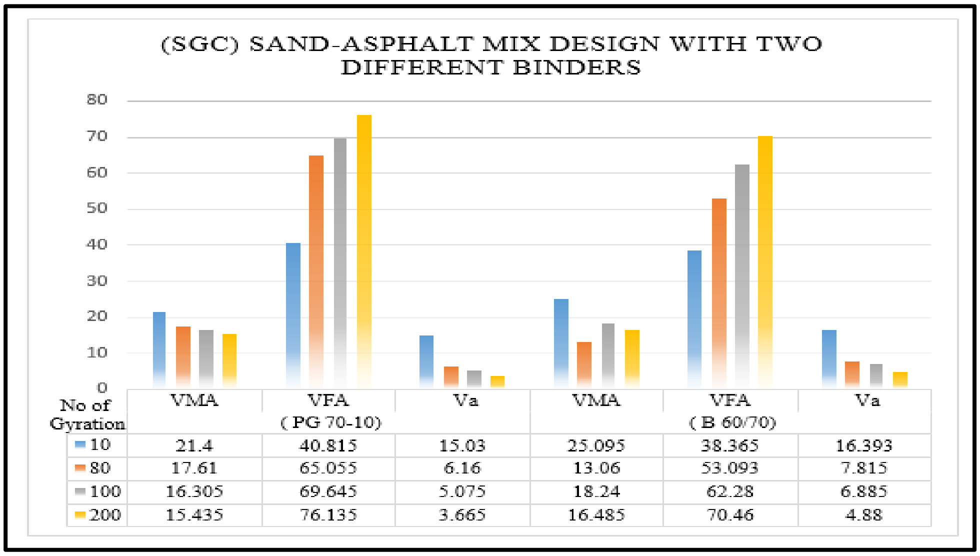


Figure AII. 4 Marshall stability results

4.2 Results of the SUPERPAVE Gyratory Compactor (SGC)

The SGC tests were done with PG70-10 on two samples, and done on B60/70, also with two samples. See Figure 5.



No of Gyration	VMA	VFA (PG 70-10)	Va	VMA	VFA (B 60/70)	Va
10	21.4	40.815	15.03	25.095	38.365	16.393
80	17.61	65.055	6.16	13.06	53.093	7.815
100	16.305	69.645	5.075	18.24	62.28	6.885
200	15.435	76.135	3.665	16.485	70.46	4.88

Figure AII. 5 Superpave gyratory compactor results

4.3 Results of the Wheel Track

As shown in Figure 6, samples containing PG70-10 showed less rutting than samples containing B 60/70 at all stages of the rutting test (1000-30000 cycles cumulative). At 3000 cycles, the PG70-10 samples lost an average of 4.39% of their original height; in contrast, after 3000 cycles, the B60/70 samples lost an average of 8.76% of their original height.

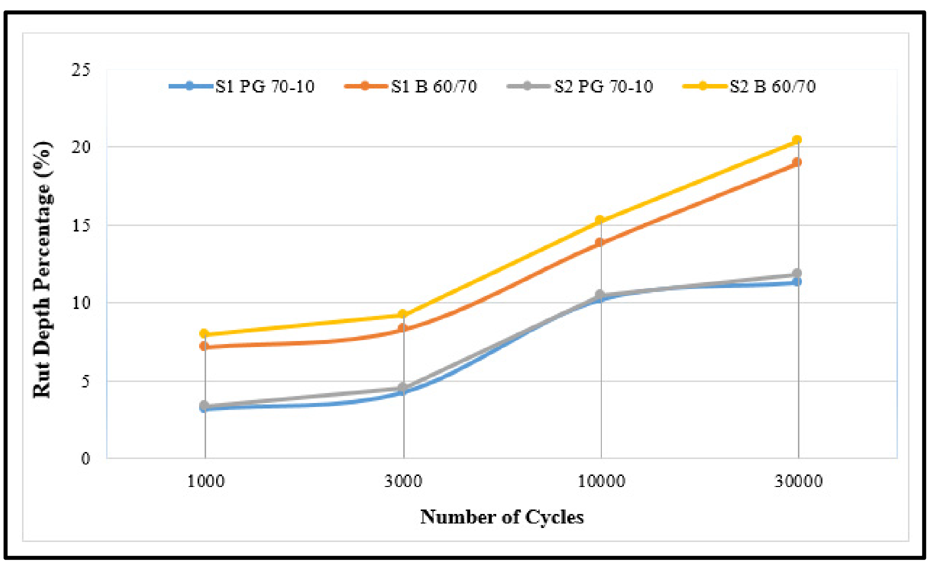


Figure AII. 6 Wheel track results

5. DISCUSSION

The first step was to determine the amount of natural sand required for all the following tests (with the bitumen). The first tests to determine the natural sand percentage were carried out with 50% natural sand content, but this gave a poor strength and stability. Following tests were carried out with 25% natural sand content and these tests gave a very good result but the project objectives were not to only find a good asphalt pavement but to make use of the available natural sand in the south of Libya, so a higher mix of natural sand was preferred; therefore, more tests were done. Following tests using 30% and 35% were done. The results of the tests with a natural sand aggregate percentage of 30% were good; the same test at 35% natural sand showed too much deformation in the HMAC. Following these tests, a 33% natural sand aggregate mix (by weight) was chosen for both HAMC for this study.

The three tests (Marshall, Gyratory Compactor, Wheel Track) indicate that the mixture of natural sand and PG70-10 performed better than a mixture of natural sand and B60/70 overall. The tests found that the mixture of natural sand and PG70-10 was generally more suitable than the mixture of natural sand and B60/70 for the hot and arid conditions under consideration.

For all the Marshall tests a fine aggregate mix of 33% natural sand and 63% manufactured aggregate was used as a control. As shown in Figure 4, the Marshall tests samples using PG70-10 all had acceptable results. The tests using B60/70 had acceptable results except for the Marshall Flow test. For this reason, PG70-10 is preferred. Also, the percentage of Air Voids (Va) in the mixture of natural sand and PG70-10 was within the acceptable limit of 4%, but the Va% of natural sand and B60/70 exceeded this recommendation. For both bitumen types, in the VMA, VFA, Va%, and Stability tests; these were all within tolerances. The Marshall Flow of the natural sand and PG70-10 mix was within the target range of 2-4 mm, but the natural sand and B60/70 Marshall Flow mix exceeded this limit according to the MTQ standard (Ministère des Transports de Quebec, 2016).

The Gyratory tests were shown in Figure 5 and illustrated that samples that contained natural sand and PG70-10 had a smaller percentage of air voids than the samples that contained natural sand and B60/70. This was true at all gyration counts. Most importantly, the samples that contained natural sand and PG70-10 had air void content within the 4-7% target range for N-des at 80 and 100 gyrations. The point that both the samples with Bitumen 60/70 exceeded the test criteria was at 80 gyrations. The other test that exceeded the tolerances was the VMA test; with the samples with Bitumen 60/70, the test for VMA exceeded the criteria set for the tests. All other tests were within tolerances (Ministère des Transports de Quebec, 2016).

In the Wheel Track test, as displayed in Figure 6, the slope of natural sand with PG70-10 changed noticeably after 10,000 cycles. From 1000 to 10,000 cycles, the slope of the natural sand with PG70-10 was comparable to the slope of natural sand with B60/70. But from 10,000 to 30,000 the test results from the natural sand with PG70-10 have a much lower slope. This is where the slope of natural sand with PG70-10 contrasts the most with the slope of the natural sand with the B60/70 results. This has important implications for a road surface in actual use. Where the natural sand with B60/70 undergoes considerable ongoing compaction, the natural sand with PG70-10 shows much more general stability over time (Ministère des Transports de Quebec, 2016).

6. CONCLUSIONS

Natural desert sand is much more worn down and therefore less angular than comparably sized manufactured aggregates; therefore, the functional characteristics of the resulting asphalt mixes are very different. The factor governing resistance to deformation in asphalt mixes are twofold: the capacity to resist shearing between aggregate and bitumen and the percentage of air voids in the bitumen mix. For these reasons, this project determined the highest practical percentage of natural desert sands that could be used in a quality HAMC for low-volume roads in high temperature remote regions of the Libyan desert; this was found to be 33%. The other factor affecting the general resistance to permanent deformation was the choice of the bitumen, necessary to create a bond between the particles. This project determined the better of two bitumen formulations, when this mix of desert sand was necessary. The recommended bitumen is PG70-10.

7. FURTHER WORK

Further research plans to investigate the performance result of different additives in the asphalt mixes. Such additives could include polymers or fibers. The information gained by these studies would provide data resources to engineers planning the next generation of road construction in hot and arid countries. The result will be the construction of better-performing roads with lower maintenance costs.

8. ACKNOWLEDGEMENTS

The first author wishes to thank the Department of Civil and Construction Engineering at (ETS), for providing the laboratory equipment for this research.

APPENDIX III

EDUCATION

- **Doctor of Philosophy (Ph.D.) in Civil and Construction Engineering**. École de Technologie Supérieure (ÉTS). Supervisor: Prof. Gabriel J. Assaf From September 2015 to June 2020. Montréal, QC – Canada.

- **Master of Science (Msc.) in Civil and Construction Engineering**, University Kebangsaan Malaysia, (UKM). Supervisor: Prof. Riza Atiq Rahmat. From August 2005 to January 2007. Malaysia.

- **Bachelor of Civil Engineering**, Tripoli University, From September 1998 to June 2003. Libya.

AWARDS AND SCHOLARSHIPS

- 2016, Libyan Student Excellence Award. Selected as a One of the nominees for the Canadian Bureau for International Education (CBIE).

- 2014, Six years of ETS scholarship.

APPENDIX IV

PERSONAL PUBLICATION LIST

JOURNAL PUBLICATIONS

- **Fathi Almadwi**, Gabriel J. Assaf (2019). " Effects of asphalt binders on pavement mixtures using an optimal balance of desert sand." Journal of Construction and Building Materials (Elsevier).

- **Fathi Almadwi**, Gabriel J. Assaf (2019). " Effect of natural desert sand on permanent deformation behaviour of Superpave asphalt mixture." Structural Engineering and Mechanics (SEM, Techno press).

- **Fathi Almadwi**, Gabriel J. Assaf (2019). " Effects of brick powder on the properties of hot mix asphalt." Journal of Materials in Civil Engineering (ASCE).

CONFERENCE PUBLICATION

- **Fathi Almadwi,** Gabriel J. Assaf (2017). Performance testing of paving mixes for Libya's hot and arid conditions, using Marshall stability and Superpave gyratory compactor methods. Advancement in the Design and Performance of Sustainable Asphalt Pavements, International Congress and Exhibition on Sustainable Civil Infrastructures. Springer 2017.

- **Fathi Almadwi,** Gabriel J. Assaf (2018). Finding an optimal bitumen and natural sand balance for hot mix asphalt concrete in hot and arid regions. In Civil Infrastructures Confronting Severe Weathers and Climate Changes Conference. Springer, Cham 2018.

BIBLIOGRAPHY

Abdulaziz A. Bubshait. (2001). Quality of Pavement Construction in Saudi Arabia. *Practice Periodical on Structural Design and Construction*, *6*(3), 129–136. https://doi.org/10.1061/(ASCE)1084-0680(2001)6

Ahlrich, R. C. (1991). The Effects of Natural Sands on Asphalt Concrete Engineering Properties. Vicksburg, MS: U.S.: Army Corps of Engineers Waterways Experiment Station.

Ahmad, J., Rahman, M. A., & Hainin, M. R. (2011). Rutting Evaluation of Dense Graded Hot Mix Asphalt Mixture. *International Journal of Engineering & Technology (IJET-IJEN)*, *11*(05), 56–60.

Ahmed, A., & Othman, R. (2000). The Evaluation of Pavement in Desert Region in Libya. In *In 2 nd Eurasphalt&Eurobitume Congress 2000,(The World Road Association, PIARC)* (pp. 500–502).

Ahmed, N. Y. (2016). Effect of Natural Sand Percentages on Fatigue Life of Asphalt Concrete Mixture, (1), 258–265.

Airey, G. D. (2003). Rheological properties of styrene butadiene styrene polymer modified road bitumens q, *82*, 1709–1719. https://doi.org/10.1016/S0016-2361(03)00146-7

Al-Hassan, M. S. A.-H. (1993). *Factors Affecting Quality of Pavement Construction in Saudai Arabia*. King Fahd university of Petroleum & Minerals Dhahran, Saudi Arabia.

Al-Juraiban, S. A., & Jimenez, R. A. (1983). Rutting Properties of a Dune-Sand Paving Mixture. *Transportation Research Record 911*.

Ali and Ezeah. (2017). Framework for Management of Post-Conflict Waste in Libya, *13*(5), 32–49. https://doi.org/10.19044/esj.2017.v13n5p32

Almadwi and Assaf. (2018). Finding an Optimal Bitumen and Natural Sand Balance for Hot Mix Asphalt Concrete in Hot and Arid Regions. In *In Civil Infrastructures Confronting Severe Weathers and Climate Changes Conference* (pp. 1–12). Springer, Cham.

Almadwi, F. S., & Assaf, G. J. (2017). Performance testing of paving mixes for Libya's hot and arid conditions, using Marshall stability and SUPERPAVE gyratory compactor methods. In *In International Congress and Exhibition" Sustainable Civil Infrastructures: Innovative Infrastructure Geotechnology* (pp. 313–323). Springer, Cham. https://doi.org/10.1007/978-3-319-61908-8

Almadwi, F. S., & Assaf, G. J. (2019). Effects of asphalt binders on pavement mixtures using an optimal balance of desert sand. *Construction and Building Materials*, *220*, 415–425. https://doi.org/10.1016/j.conbuildmat.2019.06.040

Almudaiheem, J. A. (1990). Evaluation of une Sand and Asphalt Mixes Containing Different Amounts of Crusher Waste Dust. *Transportation Research Record, Journal of Transportation Research Board*, *No.1269*, 176–184.

Arora, M. G., & Kennedy, T. W. (1997). SHRP Asphalt Binder Specifications for Saudi Environment. In *In XIIIth World meeting of the International Road FederationInternational Road Federation.*

Asi, I. M. (2007). Performance evaluation of SUPERPAVE and Marshall asphalt mix designs to suite Jordan climatic and traffic conditions. *Construction and Building Materials*, *21*(8), 1732–1740. https://doi.org/10.1016/j.conbuildmat.2006.05.036

Asif, S. A., Ahmed, N., Hayat, A., Hussan, S., Shabbir, F., & Mehmood, K. (2018). Study of adhesion characteristics of different bitumen–aggregate combinations using bitumen bond strength test. *Journal of the Chinese Institute of Engineers, Transactions of the Chinese Institute of Engineers,Series A/Chung-Kuo Kung Ch’eng Hsuch K’an*, *41*(5), 430–440. https://doi.org/10.1080/02533839.2018.1490205

Asphalt Institute. (1998). *Construction of Hot Mix Asphalt Pavement* (Vol. MS-22 2n). Lexington, Kentucky, USA.

ASTM C 136. (2015). Standard Test Method for Sieve Analysis of Fine and Coarse Aggregates 1, *14*, 1–5. https://doi.org/10.1520/C0136

ASTM C127. (2012). C 127 – 04 StandardStandard Test Method for Density, Relative Density (Specific Gravity), and Absorption. *American Society for Testing and Materials*, *i*(August), 2–7. https://doi.org/10.1520/C0128-15.2

ASTM C128. (2015). C0127 - Standard Test Method for Relative Density (Specific Gravity) and Absorption of Coarse Aggregate. *ASTM International*, 5. https://doi.org/10.1520/C0127-15.2

ASTM D1188. (1998). Bulk Specific Gravity and Density of Compacted Bituminous Mixtures Using Coated Samples 1. *Practice*, *i*(Reapproved 2015), 3–6. https://doi.org/10.1520/D1188-07R15.2

ASTM D3398. (2006a). Standard Test Method for Index of Aggregate Particle Shape and Texture. *ASTM Standards*, (ISSN: 0090-3973), 1–4. https://doi.org/10.1520/D3398-00R06.2

ASTM D3398. (2006b). Standard Test Method for Index of Aggregate Particle Shape and Texture, *00*(Reapproved 2006), 1–4. https://doi.org/10.1520/D3398-00R06.2

ASTM D6927-15. (2015). Standard Test Method for Marshall Stability and Flow of Asphalt Mixtures. *American Society for Testing Materials*, *i*, 1–7. https://doi.org/10.1520/D6927-15.2

Australian Bureau of Statistics. (2012). *History of Roads in Australia*. *EPPO Bulletin* (Vol. 44). https://doi.org/10.1111/epp.12150

Badeli, S. (2018). *Evaluation of the Damage of an Asphalt Mix under Repeated Freeze-Thaw Cycles*. École de technologie supérieure.

Badeli, S., Carter, A., & Doré, G. (2018). Cold Regions Science and Technology E ff ect of laboratory compaction on the viscoelastic characteristics of an asphalt mix before and after rapid freeze-thaw cycles. *Cold Regions Science and Technology*, *146*(June 2017), 98–109. https://doi.org/10.1016/j.coldregions.2017.12.001

Basueny, A. (2016). *Complex modulus and fatigue baviour of recycled hot mix asphalt: laboratory investigation and modelling at the material levels*. ÉCOLE DE TECHNOLOGIE SUPÉRIEURE.

Benedetto, H. D., Delaporte, B., & Sauzéat, C. (2007). Three-Dimensional Linear Behavior of Bituminous Materials : Experiments and Modeling. *International Journal of Geomechanics*, *7*(2), 149–157.

Bilondi1a, M. P., Marandi, S. M., & Ghasemi2b, F. (2016). Effect of recycled glass powder on asphalt concrete modification. *Structural Engineering and Mechanics*, *59*(02), 373–385. https://doi.org/10.12989/sem.2016.59.2.373

Breakah, T. M., Bausano, J. P., Williams, R. C., Vitton, S., Breakah, T. M., Bausano, J. P., … Vitton, S. (2011). The impact of fine aggregate characteristics on asphalt concrete pavement design life, *8436*. https://doi.org/10.1080/10298430903578937

Caeterling, J. S., Di Benedetto, C., Dorée, A., & Halman, J. (2012). Technology development projects in road infrastructure: The relevance of government championing behaviour. *Technovation*, *31*, 270–283. https://doi.org/10.1016/j.technovation.2011.02.001

Caro, S., Sánchez, D. B., & Caicedo, B. (2015). Methodology to characterise non-standard asphalt materials using DMA testing: Application to natural asphalt mixtures. *International Journal of Pavement Engineering*, *16*(1), 1–10. https://doi.org/10.1080/10298436.2014.893328

Carvalho, R. L., & Schwartz, C. W. (2006). Comparisons of Flexible Pavement Designs AASHTO Empirical Versus NCHRP Project 1-37A Mechanistic – Empirical, (1947), 167–174.

Chen, M. Z., Lin, J. T., Wu, S. P., & Liu, C. H. (2011). Utilization of recycled brick powder as alternative filler in asphalt mixture. *Construction and Building Materials*, *25*(4), 1532–1536. https://doi.org/10.1016/j.conbuildmat.2010.08.005

Chiranjeevi, T., Simhachalam, R., Kumar, D. A., & Raghuram, K. B. (2012). Laboratory Evaluation of Permanent Deformation Characteristics of Bituminous Mixes Using Different Binders. In *In Proceedings of International Conference on Advances in Architecture and Civil Engineering (AARCV)* (Vol. 21, pp. 475–480).

Chowdhury, A., Button, J. W., & Grau, J. D. (2001). *Effects of Superpave restricted zone on permanent deformation*. Texas.

Collop, A. C., & Khanzada, S. (2001). Permanent Deformation In Idealised "Sand Asphalt" Bituminous Mixtures. *Road Materials and Pavement Design*, *2*(1), 7–28. https://doi.org/10.1080/14680629.2001.9689892

Corinaldesi, V., Giuggiolini, M., & Moriconi, G. (2002). Use of rubble from building demolition in mortars. *Waste Management*, *22*(08), 893–899.

Debieb, F., & Kenai, S. (2008). The use of coarse and fine crushed bricks as aggregate in concrete. *Construction and Building Materials*, *22*(05), 886–893. https://doi.org/10.1016/j.conbuildmat.2006.12.013

Di Benedetto, H., Olard, F., Sauzéat, C., & Delaporte, B. (2011). Linear viscoelastic behaviour of bituminous materials: From binders to mixes. *Road Materials and Pavement Design*, *5*(sup1), 163–202. https://doi.org/10.1080/14680629.2004.9689992

Do, H. S., & Mun, P. H. (2008). A study on engineering characteristics of asphalt concrete using filler with recycled waste lime. *Waste Management*, *28*(01), 191–199. https://doi.org/10.1016/j.wasman.2006.11.011

Doré, G. and H. K. Z. (2009). *Cold Regions Pavement Engineering*. American Society of Civil Engineers.

Epifanio, E. G., & Gan, L. I. U. (2009). *Validation of a Pavement Performance Model for Flexible Pavements Based on Tests Results in MnROAD*. Chalmers University of Technology.

European Asphalt Pavement Association. (2008). Asphalt Pavements in Tunnels, 1–23.

Federal Aviation Administration. (2013). Project Organization, Mix Design, and Quality Control.

Federal Highway Administration. (1998). *Superpave Fundamentals NHI Course #131053*.

Federal Highway Administration. (2001). *HMA Pavement Mix Type Selection Guide*. National Asphalt Pavement Association (NAPA) and the Federal Highway Administration (FHWA).

G. W. Maupin and Diefenderfer, B. K. (2006). *Design of a High-Binder - High-Modulus Asphalt Mixture*. Virginia.

García-Morales, M., Partal, P., Navarro, F. J., Martínez-Boza, F. J., & Gallegos, C. (2007). Processing , Rheology , and Storage Stability of Recycled EVA / LDPE Modified Bitumen. *Polymer Engineering & Science*, *47*(02), 181–191. https://doi.org/10.1002/pen

Golalipour, A., Jamshidi, E., Niazi, Y., Afsharikia, Z., & Khadem, M. (2012). Effect of Aggregate Gradation on Rutting of Asphalt Pavements. *Procedia - Social and Behavioral Sciences*, *53*, 440–449. https://doi.org/10.1016/j.sbspro.2012.09.895

Guettala, S., & Mezghiche, B. (2011). Compressive strength and hydration with age of cement pastes containing dune sand powder. *Construction and Building Materials*. https://doi.org/10.1016/j.conbuildmat.2010.09.026

Gupta Ankit, P. K. and R. R. (2011). Effect of Environmental Factors on Flexible Pavement Performance Modeling. In *8th International Conference on Managing Pavement Assets EFFECT*.

Hamed M. H. Alani, A. H. A. and A. S. A. (2010). The Transition to a PG Grading System for Asphalt Cement in Iraq. *Journal of Engineering*, *16*(4).

Hassan Y. Ahmed, A. M. O. and A. A. M. (2006). Effect of Using Waste Cement Dust as a Mineral Filler on the Mechanical Properties of Hot Mix Asphalt. *Ass. Univ. Bull. Environ. Res*, *9*(1), 51–60.

Hay, Richard , E., & Peter, A. K. (1986). Relationship of Quality Control to Pavement Performance. Solutions for Pavement Rehabilitation Problems, 203-219.

Highway Research Board. (1949). *Symposium on Asphalt Paving Mixtures*. Washington DC.

Hoang, N., Cédric, P., Di, H., Juan, B., Barreto, G., Nicolaï, A., & Jakubowski, M. (2015). Analysis and modeling of 3D complex modulus tests. *Mech Time-Depend Mater*, 167–186. https://doi.org/10.1007/s11043-015-9258-8

Huang, Y., Bird, R. N., & Heidrich, O. (2007). A review of the use of recycled solid waste materials in asphalt pavements. *Resources, Conservation and Recycling*, *52*(01), 58–73. https://doi.org/10.1016/j.resconrec.2007.02.002

Hveem, F. N. (1938). Progress in Research as Applied to Maintenance of Bituminous Surfaces. In *In Proc. Montana National Bit.* (p. 160).

Islam, M. R., & Tarefder, R. A. (2015). Coefficients of Thermal Contraction and Expansion of Asphalt Concrete in the Laboratory. *Journal of Materials in Civil Engineering*, *27*(11). https://doi.org/10.1061/(ASCE)MT

Janoo V. and Korhonen C. (1999). *Performance Testing of Hot- Mix Asphalt Aggregates*. *US Army Gorps of Enginners*.

Kallas, B. F., Puzinauskas, V. P., & Krieger, H. C. (1962). Mineral Fillers in Asphalt Paving Mixtures. *Highway Research Board Bulletin*, *329*. Retrieved from http://onlinepubs.trb.org/Onlinepubs/hrbbulletin/329/329-002.pdf

Kandhal, P. S., & Khatri, M. A. (1991). *Evaluation of asphalt absorption by mineral aggregates*.

Khiavi, A. K., & Mansoori, S. (2017). The performance of hot mix asphalt in dynamic and static creep tests. *Petroleum Science and Technology*, *35*(15), 1627–1634. https://doi.org/10.1080/10916466.2017.1336773

Kim ', Y. R., Kim, N., & Khosla, N. P. (1992). Effects of Aggregate Type and Gradation on Fatigue and Permanent Deformation of Asphalt Concrete. *In Effects of Aggregates and Mineral Fillers on Asphalt Mixture Performance. ASTM International.* Retrieved from www.aslm.org

Lee, C. J., White, T. D., & West, T. R. (1999). *EFFECT OF FINE AGGREGATE ANGULARITY ON ASPHALT MIXTURE PERFORMANCE*.

Lin, C. O. N. G., Xiaoguang, Z. H. E. N. G., & Weimin, L. V. (2006). Effect of Clay Content in Fine Aggregate on Water Stability of Asphalt Mixture. *Journal of Tongji University (Natural Science)*, *5*.

Ma, F., Sha, A., Lin, R., Huang, Y., & Wang, C. (2016). Greenhouse Gas Emissions from Asphalt Pavement Construction : A Case Study in China. *International Journal of Environmental Research and Public Health*, *13*(03), 351. https://doi.org/10.3390/ijerph13030351

Mahan, H. M. (2013). Behavior of Permanent Deformation in Asphalt Concrete Pavements Under Temperature Variation. *Al-Qadisiya Journal For Engineering Sciences*, *6*(1), 62–73.

Mahboub, K. (1991). Asphalt Concrete Creep as Related to Rutting. *Journal of Materials in Civil Engineering, 2*(3), 147–163.

Mangiafico, S., Di Benedetto, H., Sauzéat, C., Olard, F., Pouget, S., & Planque, L. (2014). New method to obtain viscoelastic properties of bitumen blends from pure and reclaimed asphalt pavement binder constituents. *Road Materials and Pavement Design*, *15*(2), 312–329. https://doi.org/10.1080/14680629.2013.870639

Marks, V. J., Monroe, R. W., & Adam, J. F. (1990). *The Effects of Crushed Particles in Asphalt Mixtures (No. MLR-88-16)*.

Marques, V. D. C., de Queiroz, B. O., de Lacerda, D. M., Gouveia, A. M. D. A., & de Melo, R. A. (2014). Mechanical Performance of Asphalt Mixtures with Natural Aggregates and Recycled Aggregates for Surface Course. *In Key Engineering Materials*, *600*, 657–666. https://doi.org/10.4028/www.scientific.net/KEM.600.657

McAsphalt Industries Limited. (2012). *Basic Asphalt Materials Mixture Design and Testing*.

Mills-beale, J., & You, Z. (2010). The mechanical properties of asphalt mixtures with Recycled Concrete Aggregates. *Construction and Building Materials*, *24*(3), 230–235. https://doi.org/10.1016/j.conbuildmat.2009.08.046

Ministère-des-Transports-de-Quebec. (2016). *Norme1 LC26* (Vol. VII 4202,). Quebec, Canada.

Naceri, A., & Hamina, M. C. (2009). Use of waste brick as a partial replacement of cement in mortar. *Waste Management*, *29*(8), 2378–2384. https://doi.org/10.1016/j.wasman.2009.03.026

Niazi, Y., & Mohammadi, M. (2003). The Effect of Using Natural Sand On the Properties and Behavior of Asphalt Paving Mixes. *Journal of Faculty of Engineering (University of Tehran)*, *37*, 311–321.

Olard, F. (2012). GB5 mix design : high-performance and cost- effective asphalt concretes by use of gap-graded curves and SBS modified bitumens. *Road Materials and Pavement Design*, *13*((sup1)), 234–259. https://doi.org/10.1080/14680629.2012.657074

Olard, F., & Di Benedetto, H. (2003). General "2S2P1D" Model and Relation Between the Linear Viscoelastic Behaviours of Bituminous Binders and Mixes. *Road Materials and Pavement Design*, *4*(2), 185–224. https://doi.org/10.1080/14680629.2003.9689946

Ongel, A., Harvey, J., & Kohler, E. (2007). *State of the Practice in 2006 for Open-Graded Asphalt Mix Design*.

Park and Lee. (2002). Test methods for fine aggregate angularity considering resistance of rutting. *KSCE Journal of Civil Engineering*, *6*(4), 421–427. https://doi.org/10.1007/BF02841996

Perdomo, D., & Button, J. W. (1991). *Identifying and Correcting Rut-Susceptible Asphalt Mixtures*.

Pérez, I., & Gallego, J. (2010). Rutting prediction of a granular material for base layers of low-traffic roads. *Construction and Building Materials*, *24*(03), 340–345. https://doi.org/10.1016/j.conbuildmat.2009.08.028

Pouget, S., Sauzéat, C., Di Benedetto, H., & Olard, F. (2010). Numerical simulation of the five-point bending test designed to study bituminous wearing courses on orthotropic steel bridge. *Materials and Structures/Materiaux et Constructions*, *43*(3), 319–330. https://doi.org/10.1617/s11527-009-9491-1

Ramirez Cardona, D. A., Pouget, S., Di Benedetto, H., & Olard, F. (2015). Viscoelastic Behaviour Characterization of a Gap-graded Asphalt Mixture with SBS Polymer Modified Bitumen. *Materials Research*, *18*(2), 373–381.

Reza, M., Nayyeri, M., Niazi, Y., & Jalili, M. (2011). The Effect of Dune Sands on Permanent Deformation Characteristics of Asphalt Mixtures. *Transportation Research Journal*, *1*(1), 47–59.

Roberts, F. L., Kandhal, P. S., Brown, E. R., Lee, D. Y., & Kennedy, T. W. (1996). *Hot Mix Asphalt Material Mixture Design and Construction* (Second Edi). National Center for Asphalt Technology, NAPA Education Foundation, Baltimore, Maryland.

Salem, H. A., Uzelac, D., & Matic, B. (2014). Temperature Zoning of Libya Desert for Asphalt Mix Design. *Applied Mechanics and Materials*, *638–640*, 1414–1426. https://doi.org/10.4028/www.scientific.net/AMM.638-640.1414

Shiboub and Assaf. (2019). A System Dynamic Model for Sustainable Construction and Demolition Waste Recycling in Libya. *European Scientific Journal*, *15*(6). https://doi.org/10.19044/esj.2019.v15n6p191

Soenen, H., Redelius, P., & de La Roche, C. (2003). Fatigue Behaviour of Bituminous Materials: From Binders to Mixes. *Road Materials and Pavement Design*, *4*(1), 7–27. https://doi.org/10.1080/14680629.2003.9689938

Sousa, J., Craus, J., & Monismith, C. (1991). Summary report on permanent deformation in asphalt concrete. *National Research Council Wachington, DC*. Retrieved from http://trid.trb.org/view.aspx?id=350771

Sun, D., & Lu, W. (2007). Phase Morphology of Polymer Modified Road Asphalt. *Petroleum Science and Technology*, *24*(07), 839–849. https://doi.org/10.1081/LFT-200043780

Tam, V. W. Y., & Tam, C. M. (2006). A review on the viable technology for construction waste recycling. *Resources, Conservation and Recycling*, *47*(03), 209–221. https://doi.org/10.1016/j.resconrec.2005.12.002

Tapkın, S. (2008). Mechanical evaluation of asphalt – aggregate mixtures prepared with fly ash as a filler. *Canadian Journal of Civil Engineering*, *35*(01), 27–40. https://doi.org/10.1139/L07-082

Theyse, H. L., De Beer, M., & Rust, F. C. (1996). Overview of South African Mechanistic. *Journal of the Transportation Research Board*, *1539*(01), 6–17.

Topal, A., & Sengoz, B. (2005). Determination of fine aggregate angularity in relation with the resistance to rutting of hot-mix asphalt. *Construction and Building Materials*, *19*(02), 155–163. https://doi.org/10.1016/j.conbuildmat.2004.05.004

Transportation Research Board. (2005). *SUPERPAVE Performance by Design*. Washington DC.

Transportation Research Board. (2011). *A Manual for Design of Hot Mix Asphalt with Commentary*. WASHINGTON, D.C.

Vincent C. Janoo. (1998). *Quantification of Shape, Angularity, and Surface Texture of Base Course Materials*. *US Army Gorps of Enginners*.

Wahhab, H. I. A., Fatani, M. N., Noureldin, A. S., & Bubshait, A. (1995). National Study of Asphalt Pavement Rutting in Saudi Arabia. *Transportation Research Record*, (1473).

Wong, Gordon, L. and A. (2016). Effect of Fine Aggregate Composition on Moisture Susceptibility of Hot Mix Asphalt. In *8th RILEM International Symposium on Testing and Characterization of Sustainable and Innovative Bituminous Materials* (pp. 735–744). Springer, Dordrecht.

Yang H. Huang. (2004). *Pavement Analysis and Design* (Second).