EFFECTS OF MANUFACTURED FINE AGGREGATE ON PHYSICAL AND MECHANISTIC PROPERTIES OF SASKATCHEWAN ASPHALT CONCRETE MIXES

A Thesis Submitted to the College of
Graduate Studies and Research
in Partial Fulfillment of the Requirements
for the Degree of Master of Science
in the Department of Civil and Geological Engineering
University of Saskatchewan
Saskatoon

By
Anna Maria Anthony, P. Eng.

PERMISSION TO USE

In presenting this thesis in partial fulfillment of the requirements for a Postgraduate degree from the University of Saskatchewan, the author has agreed that the Libraries of this University may make it freely available for inspection. The author has further agreed that permission for copying of this thesis in any manner, in whole or in part, for scholarly purposes may be granted by the professor or professors who supervised the thesis work or, in their absence, by the Head of the Department or the Dean of the College in which the thesis work was done. It is understood that any copying, publication, or use of this thesis or parts thereof for financial gain shall not be allowed without the author's written permission. It is also understood that due recognition shall be given to the author and to the University of Saskatchewan in any scholarly use which may be made of any material in this thesis.

Requests for permission to copy or to make other use of material in this thesis in whole or part should be addressed to:

Head of the Department of Civil Engineering

University of Saskatchewan

Saskatoon, Saskatchewan S7N 5A9

ABSTRACT

Saskatchewan Highways and Transportation (SDHT) rely on dense-graded hot mix asphalt concrete mixes for construction and rehabilitation of asphalt pavement surfaced highways. As a result of increased commercial truck traffic on the provincial road network, over the last two decades, some of Saskatchewan's recently placed dense graded hot mix asphalt concrete (HMAC) pavements have been observed to show a susceptibility to premature permanent deformation in the asphalt mix. One of the aggregate properties thought to have significant influence on mix performance under traffic loading is the shape of the aggregate. Specifically, the physical properties of the fine aggregate (smaller than 5 mm in diameter) are of particular importance in dense graded mixes. Although empirical evidence suggests that there are performance benefits associated with using angular fine aggregate, the relationship of this parameter on mechanistic mix performance and resistance to permanent deformation has not yet been clearly defined.

The primary objective of this research was to conduct laboratory analysis to determine the physical, empirical, and mechanistic behaviour sensitivity to the proportion of manufactured and natural fine aggregate in SDHT Type 72 hot mix asphalt concrete. The second objective of this research was to compare the mechanistic behaviour of the Type 72 mixes considered in this research to conventional SDHT Type 70 structural hot mix asphalt concrete.

Physical and mechanistic properties of a SDHT Type 72 mix at levels of 20, 40, and 60 percent manufactured fines as a portion of total fines (smaller than 5 mm), and for a SDHT Type 70 mix (which contained 38 percent manufactured fines) were evaluated. Ten repeat samples were compacted for each mix using 75-blow Marshall compaction, and ten samples for each mix were compacted using the Superpave™ gyratory compaction protocols. Marshall stability and flow testing was conducted on the Marshall-compacted samples. Triaxial frequency sweep testing was conducted on the gyratory-compacted samples using the Rapid Triaxial Tester (RaTT) at 20°C. The testing was conducted at axial loading frequencies of 10 and 0.5 Hz, and at deviatoric

stress states of 370, 425, and 500 kPa, respectively. The resulting dynamic modulus, axial and radial microstrains, Poisson's ratio, and phase angle were evaluated.

The research hypothesis stated that the increased amount of manufactured fines improves mechanistic properties of the Type 72 mix under typical field state conditions, and Type 72 mix with increased manufactured fines can exhibit mechanistic properties equivalent to or exceeding those of a typical type 70 mix.

Based on the improved densification properties, increased Marshall stability, increased dynamic modulus, and reduced radial and axial strains, it was demonstrated that increasing manufactured fines content in the SDHT Type 72 mix does improve the mechanistic properties of this dense-graded asphalt mix. It should be noted that there appears to be a minimum level of manufactured fines content that is required to affect mix response to loading, and that this threshold lies somewhere between 40 and 60 percent manufactured fines content for the Type 72 mix tested as part of this research.

Further, the Type 72 mix exhibited comparable or improved mechanistic properties relative to the Type 70 mix, which SDHT consider a structural mix. This is illustrated by the Type 72 mix with 60 percent manufactured fines resulting in higher Marshall stability and dynamic modulus, and lower axial microstrains than the Type 70 mix evaluated in this study.

It is recommended that other Type 72 and Type 70 mixes are evaluated using similar testing protocols. In addition, field test sections should be used to further verify the research hypothesis investigated here.

Economic analysis indicates that substantial savings in life cycle costs of SHDT asphalt concrete surfaced roadways can be realized by engineering well-performing, rut-resistant mixes. The life cycle costs can be reduced annually by approximately $7.4 million, which translates into $102.5 million savings over 18 years, during which the entire pavement network would be resurfaced with well-performing asphalt concrete mixes.

Further, enhanced crushing of smaller aggregate top size decreases the amount of rejected material, and increases manufactured fines to coarse aggregate ratio, resulting not only in better engineering properties, but also in the optimized use of the province's diminishing gravel resources. Pressures on aggregate sources are also reduced by improving life cycle performance of Saskatchewan asphalt concrete pavements. The total potential aggregate savings that can be realized by implementing well-performing Type 72 HMAC mixes amount to 4.3 million metric tonnes of aggregate in the next 42 years. These aggregate savings can help decrease the predicted shortage of aggregate between 2007 and 2049 by approximately 6 percent. The total potential cost savings after 18 years of paving 500 km per year with rut-resistant, well-performing HMAC mixes amount to $112.4 million in present value dollars. The 42 year savings amount to $193.7 million in present day dollars. It is recommended that a more detailed economic analysis be carried out.

ACKNOWLEDGEMENTS

During the preparation of this thesis, the author was fortunate to receive assistance from several dedicated engineers, professors, technicians, researchers, family members, and friends.

The encouraging words from the late Dr. E. K. Sauer are what sparked the author's interest in this professional journey, while the support of Saskatchewan Highways and Transportation leadership team, in particular that of Daryl Nixon, Gordon King, and Doug Hansen, was crucial towards its successful conclusion. The assistance of Saskatchewan Highways and Transportation laboratory staff, especially that of Ron Genest, Kerry Bray, and Russ Gowan, was essential toward the completion of the extensive laboratory testing plan set out for this study. The aid from Glen Whitford, Brent Marjerison, Marc Zintel, Josh Safronetz, Wendell Peterson, Iftekhar Kalyar, Andrea Bueckert, and numerous other SDHT staff, as well as from Ciprian Raducanu and Rob Bushman, is also gratefully acknowledged.

The successful completion of this research project would not have been possible without the continued guidance and encouragement received from the author's research supervisor, Dr. Curtis Berthelot. The author would also like to thank the advisory committee for their input: Dr. Bruce Sparling, Dr. Gordon Sparks, Dr. Dennis Pufahl, and Mr. Greg Chartier.

The achievement of a Master of Science degree is so far the author's largest and longest professional undertaking, requiring enormous supplies of patience and motivation. Heartfelt gratitude is extended to the author's family and friends who over the years continued to grant patience, understanding, support and encouragement. Especially, the love and support of husband Jeffrey, parents Zenon and Marianna, and brother Gregory, have been essential on this journey.

Special thanks are extended to Saskatchewan Highways and Transportation for funding this research, and for recognizing the need for innovation in the application of materials science principles in the area of pavement design and analysis.

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

LIST OF ABBREVIATIONS

AASHTO – American Association of State Highway and Transportation Officials
ANOVA – Analysis of variance
ASTM – American Society for Testing and Materials
CGSB – Canadian General Standards Board
COS – City of Saskatoon
CV – Coefficient of variation
E* – Complex modulus
E_d – Dynamic modulus
EPS – End Product Specification
ESAL – Equivalent Single Axle Load
FAA – Fine Aggregate Angularity
FHWA – Federal Highway Administration
G_{mm} – Theoretical maximum specific gravity
G_{sb} – Bulk specific gravity of aggregate
HMAC – Hot Mix Asphalt Concrete
IDT – Indirect Tension Test
LCPC – Laboratoire Central des Ponts et Chausseés
LOC – Level of confidence
LVDT – Linear Variable Differential Transducer
N_{des} or N_{design} – Design number of gyrations
N_{ini} or $N_{initial}$ – Initial number of gyrations
N_{max} or $N_{maximum}$ – Maximum number of gyrations
OGFC – Open Graded Friction Course
PG – Performance Grade
PV – Present Value
RAMS – Recoverable Axial Microstrain
RRMS – Recoverable Radial Microstrain
RaTT – Rapid Triaxial Tester
SA – Surface area of aggregate (m^2 per Kg of aggregate)
SD – Standard Deviation
SDHT – Saskatchewan Department of Highways and Transportation
SSD – Saturated Surface-Dry
SHRP – Strategic Highway Research Program
SMA – Stone Mastic Asphalt
SST – Superpave™ shear tester
STP – Standard Test Procedure
Superpave™ – SUperior PERforming asphalt PAVEments
T_{eff} – Effective temperature
T_f – Average film thickness
V_{EAC} – Volume of effective asphalt content
VFA – Voids Filled with Asphalt
VMA – Voids in the Mineral Aggregate
VTM – Voids in Total Mix

CHAPTER 1 INTRODUCTION

Saskatchewan Highways and Transportation (SDHT) currently operate and maintain 8,975 km of structural hot mix asphalt concrete (HMAC) pavements (Kalyar 2005). As a result of increased commercial truck traffic on the provincial road network, over the last two decades, some of Saskatchewan's recently placed dense graded HMAC pavements have been observed to show a susceptibility to premature permanent deformation in the form of rutting in the asphalt mix (Huber and Heiman 1987, Carlberg *et al.* 2002, SDHT 2003-A). This problem is significantly decreasing the expected in-service life of the affected pavements thus creating a concern for long term sustainability of the highway infrastructure.

Saskatchewan is not the only agency experiencing premature permanent deformation problems. Higher traffic volumes, increased loads and decreasing aggregate quality in many jurisdictions, have resulted in premature rutting becoming a problem for many road authorities in North America (Brown and Cross 1992), and significant resources have been directed toward creating long lasting pavements (Asphalt Institute 1996).

To achieve more structural, rut resistant mixes, SDHT implemented a 75 blow Marshall mix design, replacing the traditional 50 blow design on the National Highway System in 1999, and increased coarse aggregate fracture requirements for all SDHT mixes. This increase in mix design standards resulted in the use of coarser aggregate gradations, which has increased aggregate costs and accelerated aggregate source usage. In addition, these coarser Saskatchewan HMAC mixes have also become more sensitive to handling and placement.

Construction problems such as segregation and difficulties in achieving compaction associated with the placement of coarse mixes have resulted in

Saskatchewan contractors requesting the use of finer HMAC mixes as surface course. Contractors reason that finer mixes are more workable and less sensitive to handling, therefore reducing the potential for placement problems, especially segregation, and improving the visual quality as well as the durability of the finished surface. The contractors' interest in improving surface quality relates directly to the segregation, compaction, and ride and roughness penalties imposed by SDHT.

Although utilizing smaller top size and finer aggregate would improve the HMAC pavements construction process, there is a concern that finer mixes may be more susceptible to permanent deformation under heavy vehicle loading due to their reduced aggregate skeleton. In light of already substantial problems with premature permanent deformation, there is a need to determine the performance feasibility of using finer mixes, without further increasing the potential for permanent deformation, while at the same time mitigating the susceptibility to permanent deformation.

One of the aggregate properties thought to have significant influence on mix performance under traffic loading is the shape of the aggregate (Brown and Cross 1992, Button *et al.* 1990). Angular rocks are thought to provide better stone on stone interlock than rounded aggregate, therefore reducing the susceptibility to rutting (Asphalt Institute 1996, Ahlrich 1996, Marks *et al.* 1990). Further, the physical properties of the fine aggregate (smaller than 5 mm in diameter) are of particular importance in dense graded mixes, because the coarse aggregates (greater than 5 mm in diameter) are usually not in contact with each other, rather, they are suspended in the fine aggregate, which is forced to carry the load (Roberts *et al.* 1996, Perdomo *et al.* 1992, Parker and Brown 1992). Although empirical evidence suggests that there are performance benefits associated with using angular fine aggregate, the relationship of this parameter on mechanistic mix performance and resistance to permanent deformation has not yet been clearly defined.

1.1 Research Goal

The goal of this research project is to improve the field performance of hot mix asphalt pavements in Saskatchewan.

1.2 Importance of Research

Investigating the influence of manufactured fines on the conventional and mechanistic properties of Saskatchewan hot mix asphalt concrete mixes is important to the province of Saskatchewan for several reasons, some of which are listed below:

- With limited funding and increased budget pressures in the province, providing well-performing asphalt pavements is critical toward supporting the provincial economy, and sustaining efficient and effective transport in the province.
- In light of the documented rutting problems in the province, improving rutting performance of Saskatchewan mixes would reduce the amount of preservation funds required to fill in premature ruts, and extend the period between initial construction and first rehabilitation.
- In addition to the operations and preservation problems directly related to permanent deformation, the presence of ruts also contributes to increased severity and acceleration of other distresses. Engineering rut-resistant mixes should also decrease pavement susceptibility to other distresses and increase pavement life.
- Rutted pavements pose a safety concern for the road users, resulting in increased user, social, and agency operating costs. Reducing rutting susceptibility could potentially increase the safety of the road user and therefore reduce the society's costs associated with highway collisions and fatalities.
- If increasing manufactured fines outweighs the benefits of larger top size of aggregate, crushing of smaller top size of aggregate may decrease the amount of rejected material, better utilizing the province's diminishing gravel resources.
- Increasing coarse aggregate angularity and implementing coarser aggregate specifications for hot mix asphalt concrete mixes further increases pressures on non-renewable aggregate resources. Improving the performance of

pavements and therefore increasing their life cycle can result in the decrease of volume of material required annually.

- Given the limitations of conventional Marshall properties to accurately predict rutting, characterizing the mechanistic properties of SDHT asphalt mixes is one of the necessary steps towards implementing performance-related structural parameters in the SDHT asset management system.

- With Saskatchewan aggregate being manufactured from glacial gravel deposits, it is current practice to incorporate natural sands in hot mix aggregate gradations. Determining the sensitivity of SDHT mixes to the amount of natural versus manufactured fines content with respect to the physical and mechanistic properties of the mixes is necessary to provide insight into maximizing aggregate source utilization, by incorporating natural fines into the hot mix aggregate without compromising field performance.

- Although using finer mixes may be a feasible solution for the asphalt pavement contractors to reduce penalties, it may, in the long term, result in increased network management and user costs, if permanent deformation susceptibility is not investigated and mitigated within the specified material constitutive relations of SDHT dense graded HMAC mixes.

- SDHT does not control the proportions of natural and manufactured fine aggregate for hot mix asphalt, nor are any physical properties that would address fine particle shape included in current specifications. It is therefore not known how SDHT dense graded mixes will perform at various levels of manufactured fine aggregate content.

Permanent deformation is a problem that continues to affect not only Saskatchewan, but the entire flexible pavement engineering community (Sousa *et al.* 1991), and understanding the mechanistic material constitutive properties as a function of the various field state conditions is the first step towards being able to confidently predict pavement performance.

1.3 Research Objectives

The primary objective of this research has been to conduct laboratory analysis to determine the physical and mechanistic behaviour sensitivity to the proportion of manufactured and natural fine aggregate in SDHT Type 72 hot mix asphalt concrete. A second objective of this research is to compare the mechanistic behaviour of the Type 72 mixes considered in this research to conventional SDHT Type 70 structural hot mix asphalt concrete.

1.4 Research Hypothesis

It is hypothesised that the increased amount of manufactured fines improves mechanistic properties of the Type 72 mix under typical field state conditions. It is also hypothesized that Type 72 mix with increased manufactured fines can exhibit mechanistic properties equivalent to or exceeding those of a typical type 70 mix.

1.5 Scope

Three Type 72 HMAC mixes were considered for this research, based on a mix design used for a SDHT pavement rehabilitation project of Highway 11, south of Craik (Contract No. M01091). The aggregate blends had 20, 40, and 60 percent of manufactured fine aggregate, respectively, as determined by weight on the portion of total fine aggregate within the mix (passing the 5 mm sieve). The amount of manufactured coarse aggregate (retained on the 5 mm sieve) was maintained constant across the Type 72 mixes considered, and the manufactured fines were substituted for natural fines in order to vary the manufactured fines content only. The structural Type 70 mix used in the study had 38 percent fine aggregate, as manufactured for the above mentioned Highway 11 paving project. All HMAC samples were created with 150/200A straight run asphalt cement and 0.7 percent of liquid anti-stripping agent by weight of asphalt cement.

The laboratory characterization involved assessing volumetric properties using the standard 75 blow Marshall mix design method, as well as SHRP Level 1 gyratory compaction. Marshall stability and flow were determined, and triaxial frequency sweep testing was performed across various frequencies and stress states at 20°C.

1.6 Methodology

The following project elements and tasks were employed in this research:

- Project Element 1: Background and Literature Review.
  - Task 1 - Literature review of previous research investigating the effect of manufactured fines and aggregate properties on the physical and mechanistic performance of hot mix asphalt concrete.
  - Task 2 - Review of SDHT specifications for HMAC and the mix design process.
- Project Element 2: Material Sampling.
  - Task 1 - Aggregate sampling from Contract No. M01091.
  - Task 2 - Asphalt cement sampling from same supplier and of the same grade (150/200A) as used on the Hwy 11 construction project.
  - Task 3 - Anti-stripping agent sampling from the same supplier and of the same grade as used on the Highway 11 construction project.
- Project Element 3: Sample Preparation.
  - Task 1 - 75 blow Marshall mix design (STP 204-10).
  - Volumetric analysis (STP 204-21 based on ASTM D2726).
  - Marshall stability and flow (STP 204-11 based on ASTM D1559).
  - Task 2 - SHRP Level 1 gyratory compaction (AASHTO TP-4).
  - Task 3 - Volumetric analysis of gyratory samples (STP 204-21 based on ASTM D2726).
- Project Element 4: Aggregate Characterization.
  - Task 1 - Specific Gravity (STP 206-07).
  - Task 2 - Flat and Elongated Particles (ASTM D4791).
  - Task 3 - Lightweight Pieces (STP 206-09 based on ASTM C123).
  - Task 4 - Atterberg Plasticity Index (STP 206-04 based on ASTM D4318).
  - Task 5 - Uncompacted Void Content of Fine Aggregate (ASTM C1252-03

Test Method A).

- Task 6 - Coarse Aggregate Fracture (STP 206-14).
- Task 7 - Sand Equivalent (STP 206-05 based on ASTM D2419).

➢ Project Element 5: Triaxial frequency sweep mechanistic characterization at 20°C.

- Task 1 – Laboratory testing evaluation at two load frequencies (0.5 Hz and 10 Hz) and three deviatoric stress states (370 kPa, 425 kPa, and 500 kPa).

➢ Project Element 6: Statistical Analysis consisting of summary statistics, analysis of variance (ANOVA), Tukey's pairwise comparison, and analysis of level of confidence of laboratory characterization results across independent variables stress state, temperature, and mix type at each loading frequency.

- Task 1 – Quantify relationship between amount of manufactured fines and volumetric properties of compacted gyratory samples.
- Task 2 – Quantify relationship between amount of manufactured fines and Marshall stability and flow.
- Task 3 – Quantify relationship between amount of manufactured fines and dynamic modulus.
- Task 4 – Quantify relationship between amount of manufactured fines and Poisson's Ratio.
- Task 5 – Quantify relationship between amount of manufactured fines and Recoverable Axial Microstrains.
- Task 6 – Quantify relationship between amount of manufactured fines and Recoverable Radial Microstrains.
- Task 5 – Quantify relationship between amount of manufactured fines and phase angle.

➢ Project Element 7: Economic Analysis of Implementing Type 72 Mixes.

- Task 1 – Life cycle cost analysis and determination of benefits in improved rutting performance of SHDT asphalt concrete mixes.
- Task 2 – Analysis of impacts on gravel source utilization when manufacturing Type 72 mix aggregate and Type 70 mix aggregate.
- Task 3 – Analysis of aggregate savings from the reduction of preservation

treatments during a pavement life cycle.

- Project Element 8: Summary, Conclusions and Future Research.

1.7 Layout of Report

Chapter One provides the introduction to and the significance of the work undertaken in this research. This section also includes the goal, objectives, scope and methodology relevant to this work, as well as the layout of the thesis. Chapter Two summarizes background information and previous research on issues relevant to this thesis, in context of Saskatchewan Highways and Transportation pavement mix design and specifications. The definition of permanent deformation, description of common types of HMAC mixes, brief discussion on aggregate and HMAC physical properties of relevance to pavement engineering, as well as an introduction to the Marshall, Hveem, and Superpave™ Level I mix design methods are discussed. Chapter Two also contains an introduction to the mechanistic material characterization, and specifically to repeated load triaxial frequency sweep testing. Chapter Three summarizes the conventional material properties of the research mixes that were evaluated as part of this research. The various physical aggregate properties of the research mixes, as well as the volumetric properties of each the Marshall and gyratory compacted samples are discussed. Analysis of Marshall stability and flow testing is also included. Finally, statistical significance of the results is investigated. Chapter Four reports the mechanistic material properties evaluated with the use of the triaxial frequency sweep testing, including the dynamic modulus, the recoverable portions of axial and radial microstrains, Poisson's ratio, and phase angle. The applicability of the sample size used for this research is verified based on the mechanistic test results. Chapter Five contains an economic assessment related to implementing well-performing, rut-resistant Type 72 mixes, with respect to SDHT pavement life cycle costs and provincial gravel source management. Chapter Six presents the summary, conclusions, and recommendations that can be made based on the results of this study.

CHAPTER 2 BACKGROUND AND LITERATURE REVIEW

This chapter summarizes background information related to issues relevant to this thesis, including challenges facing Saskatchewan Highways and Transportation in the area of asphalt mixes. Definition of permanent deformation in hot mix asphalt concrete pavements and description of common types of HMAC mixes, as well as a brief discussion on physical aggregate and HMAC mix properties of relevance to pavement engineering and mix performance is included. An introduction to the Marshall mix design method with reference to SDHT specifications and other mix design methods are also discussed, followed by an introduction to mechanistic characterization of hot mix asphalt concrete mixes. Limitations of empirical measures will be discussed.

2.1 Saskatchewan Highways and Transportation Challenges

Saskatchewan Highways and Transportation (SDHT) currently operate and maintain 8,975 km of hot mix asphalt concrete (HMAC) pavements (Kalyar 2005). The shift in transportation policy over the last two decades has resulted in the abandonment of branch rail lines, and prompted a large increase in commercial traffic on Saskatchewan roads (SDHT 1999). Across the entire provincial road system, annual traffic loading on the provincial pavement network increased 56 percent over the last decade, from 2.54 billion Equivalent Single Axle Loads (ESALs) in 1994, to 3.96 billion in 2004 (Anderson 2005). Other contributing factors such as an increase in trans-border trade, economic diversification, and the expectation of just-in-time delivery have also increased commercial road transportation in Saskatchewan. Along with the increase in demand for road transportation of goods, there has been an associated need for increased load capacity and more efficient truck configurations, potentially resulting in significant increase of loading on the already aged and distressed Saskatchewan highway system.

As a result of the increased commercial truck traffic on the provincial road network, some of Saskatchewan's recently placed dense graded HMAC pavements have demonstrated a susceptibility to premature permanent deformation in the asphalt mix (Huber and Heiman 1987, Carlberg *et al.* 2002, SDHT 2003-A), significantly decreasing the expected in-service life of the pavements and creating a concern for long term sustainability of the highway infrastructure. Specifically, the permanent deformation conditions on the provincial asphalt pavement road network are increasing. Between the years 2003 and 2006, provincial asphalt pavements have deteriorated from 8.3 to 11.5 percent of poor condition in terms of rutting, meaning that average rut depths across the road segments evaluated are equal to or greater than 10 mm (Kalyar 2006).

Saskatchewan Highways and Transportation have operated the provincial network for the last three years within the means of an annual budget of approximately $300 million. During the fiscal year 2005-2006, from a total budget of $307.6 million, a total of $40 million (13 percent) was spent on infrastructure rehabilitation, and $82 million (27 percent) was allocated to preservation of the transportation system (SDHT 2006). Approximately $44 million (14 percent) was spent directly on the material purchase and placement of close to 600,000 metric tonnes of hot mix asphalt concrete, used on capital and preservation road construction projects during the 2005/06 construction season.

Based on these expenditures and given the extent of the Saskatchewan road network, there is a need to ensure that the small amount of funding dedicated directly to hot mix asphalt paving is spent on quality paving products with maximized service life. With limited funding and increased budget pressures in the province, providing value-engineered asphalt pavements is critical toward supporting the provincial economy, and sustaining efficient and effective transport in the province.

Improving rutting performance of Saskatchewan mixes would reduce the amount of preservation funds required to fill premature ruts, and extend the period between initial construction and first rehabilitation required. For instance, the cost to rehabilitate the 11.5 percent of roads currently in poor rutting condition, by removing 50 mm of the rutted layer and replacing it with a new structural HMAC pavement overlay, is estimated at approximately \$103 million (Marjerison 2005). Although this is a commonly used rehabilitation approach for prematurely rutted pavements, SDHT has not conducted any studies to ensure that this treatment is in fact sufficient to improve rutting performance given current mix types used by SDHT.

SDHT designs HMAC pavements for a 15 year design life, based on projected number of Equivalent Single Axle Loads (ESALs), expecting only to invest in routine maintenance during this period (Widger 2005). However, in addition to the operations and preservation problems directly related to permanent deformation, the presence of ruts can also contribute to increased severity and acceleration of other distresses. As an example, the accumulation of moisture in wheel paths weakens the pavement structure due to water infiltration through transverse and fatigue cracks that intercept the ruts in the pavement surface. Concentrated water infiltration results in increased surface distortion and loss of structural integrity caused by water movement and freeze-thaw action within the pavement substructure. Therefore, engineered rut-resistant mixes should also decrease pavement susceptibility to other distresses and increase structural performance and pavement life.

Along with increased direct costs of rehabilitation and decreased asset life, rutted pavements pose a safety concern for the road users, therefore resulting in increased user, social, as well as agency operating costs. The longitudinal depressions in the wheel paths accumulate water, causing drivers to have reduced control of the vehicle. In the winter, ruts can accumulate ice and snow, making snow/ice removal difficult, and creating a further safety hazard. Changing lanes can also be inhibited, and since the ruts are mainly formed by heavy commercial vehicles, passenger vehicles which have a narrower wheel base may experience difficulties steering (Emery 1990). Road surface conditions caused by weather and short-term maintenance operations were listed as a contributing factor in eight fatal collisions on provincial highways in Saskatchewan in 2002 (SGI 2002).

To mitigate rutting, other North American road authorities are investigating the use of more coarse and larger top size mixes, such as Superpave™, open graded friction courses (OGFC), and stone mastic asphalt (SMA). Although SDHT has conducted trials using some of these types of mixes in the past (Siciliano and Qayyum 1994, Berthelot 1999), the agency continues to rely on three dense graded hot mix aggregate gradations, with 18 mm, 16 mm, and 12.5 mm top size, respectively. While coarse mixes often provide better rutting resistance when designed and constructed properly, they rely on large top size and highly fractured coarse aggregate to obtain the mechanical performance benefits. In light of declining availability of quality aggregate resources in the province, and challenges with limited funding, the increased requirement for larger stone, and high fracture aggregates, renders coarse mixes economically prohibitive in most cases in Saskatchewan.

With the exception of the Cypress Hills area, Saskatchewan has been glaciated at least four times (Sauer 2000). Most of Saskatchewan's highway network is located in the southern portion of the province, in an area of thick glacial deposits, and all of the HMAC aggregate is manufactured from surface glacial gravel sources. While there are aggregate-rich areas in Saskatchewan, it is becoming increasingly difficult to locate new aggregate sources, and existing quality sources suitable for HMAC aggregate production are being exhausted. Most areas in the province now require average truck hauls of 30 km or greater for delivering aggregate to construction sites, and these distances are estimated to increase for some areas by as much as 30 percent in the next fifty years (SDHT 2001-A).

Increasing coarse aggregate angularity and implementing coarser aggregate specifications for hot mix asphalt concrete mixes further increases pressures on non-renewable quality aggregate resources. SDHT has recognized these issues and has put effort into optimizing the use of existing sources, along with developing a long term aggregate management strategy (SDHT 2001-A). From this study, it is estimated that 193.3 million cubic metres of quality aggregate will be required to meet the provincial needs up to the year 2049. Based on a summary of known provincial sources at the time of the study, it is estimated that the province currently has 150 million cubic metres in available gravel sources of varying degrees of quality. Improving the performance of pavements and therefore increasing their life cycle can result in the decrease of volume of material required annually.

As an alternative to larger top size and coarser mixes, polymer-modified asphalt cement and other modified asphalt cement products are also being investigated by many agencies to increase the resistance to permanent deformation (Ponniah and Kennepohl 1996, Prowell 2001). Although this approach is worth considering, modified asphalt products cost substantially more than straight-run asphalt products, and they are more difficult to place (Brule 1996, Zubeck 2003, Better Roads 2005).

In addition to potential savings due to improved pavement performance and extended performance life cycle, if the permanent deformation resistance can be engineered in mixes with finer gradations and smaller top size aggregates, Saskatchewan will benefit from reducing aggregate wastage. For example, a typical crushing process involves screening off any natural material smaller than 9 mm, and crushing the remaining aggregate larger than 9 mm. The resulting manufactured material is usually split on the 5 mm sieve, into a manufactured fines and a manufactured coarse pile, respectively. When manufacturing coarse hot mix aggregate, it is also common practice to screen off "pea gravel" (ranging in size from 9 mm up to top size of the mix being produced), since it is thought to be too small to obtain good fracture through the crushing process. These practices can result in high quantities of rejected material, rendered useless in the hot mix aggregate manufacturing process. Enhanced crushing of smaller top size of aggregate may decrease the amount of rejected material, better utilizing the province's diminishing gravel resources.

With Saskatchewan aggregate being manufactured from glacial gravel deposits, it is current practice to incorporate natural sands in hot mix aggregate gradations. This is done partly to provide workability in the mixes, but more importantly to utilize as much of the gravel source as possible. Another mix design practice is to utilize blender/filler sands when necessary to increase or decrease the air voids in the mix to obtain the desired volumetric properties, which at this point are the primary quality control parameters of HMAC used in SDHT mix design and specifications. However, using large amounts of natural sands in the aggregate structure increases the possibility of creating "tender" mixes – ones that densify too quickly, and are therefore prone to premature plastic deformation (Hesp *et al.* 2002). Large amounts of fine aggregate also decrease asphalt cement film thickness in the mix, therefore creating a potential for moisture susceptibility and durability problems. Determining the sensitivity of SDHT mixes to the amount of natural versus manufactured fines content with respect to the physical and mechanistic properties of the mixes is necessary to provide insight to maximize aggregate source utilization, by incorporating natural fines into the hot mix aggregate without compromising field performance.

Further challenges for Saskatchewan road infrastructure management come from a lack of structural parameters in the current road asset management systems used by SDHT. To date, funding is allocated strictly based on surface condition. In striving towards a structural asset management system, there is a need to quantify the mechanistic structural properties of road materials commonly used in Saskatchewan. Although improving structural properties of SDHT asphalt mixes will not be directly measurable by the asset management tools currently in use, improving structural performance is critical given the increased traffic loadings. Characterizing the mechanistic properties of SDHT asphalt mixes is one of the necessary steps towards implementing performance-related structural parameters in SDHT asset management methods.

Although using finer mixes may be a feasible solution for the asphalt pavement contractors to reduce penalties, it may, in the long term, result in increased network management and user costs, if permanent deformation susceptibility is not investigated and mitigated. SDHT does not control the proportions of natural and manufactured fine aggregate for hot mix asphalt, nor are any physical properties that would address fine particle shape included in current specifications. It is, therefore, not known how SDHT dense graded mixes will perform at various levels of manufactured fine aggregate content.

2.2 Permanent Deformation in Flexible Pavements

Permanent deformation in asphalt (flexible) pavements, commonly referred to as rutting, usually consists of longitudinal depressions in the wheel paths, which are an accumulation of small amounts of unrecoverable deformation caused by each load application (Asphalt Institute 1996). Depending on the specific failure mode, these wheel path depressions may be accompanied by small heaves on either side. The depressions are a direct result of repeated load applications, and are caused by either densification, or shear deformation, or a combination of both, in any one or more of the pavement structural layers and/or in the subgrade (Sousa *et al*. 1991).

Based on the origin of the deformation within the road structure, rutting can be divided into two main types. The first type of rutting is a result of structural integrity problems within the road structure, including the subgrade, subbase or base. The layers are either lacking in strength or thickness to handle the applied loading, or are weakened by excess moisture. For example, if the thickness of structural surfacing with select materials is inadequate for the amount and type of traffic loading, the *in-situ* soils (subgrade) will deform due to the excessive stresses. As another example, excess fines in the granular base course or subbase layers can increase moisture attraction and retention, therefore resulting in permanent deformation. These types of rutting will result in depressions within the weak layer, and subsequent deformation of the surface asphalt layer in order to conform to the underlying distorted cross-section, as shown in Figure 2.1 and Figure 2.2.

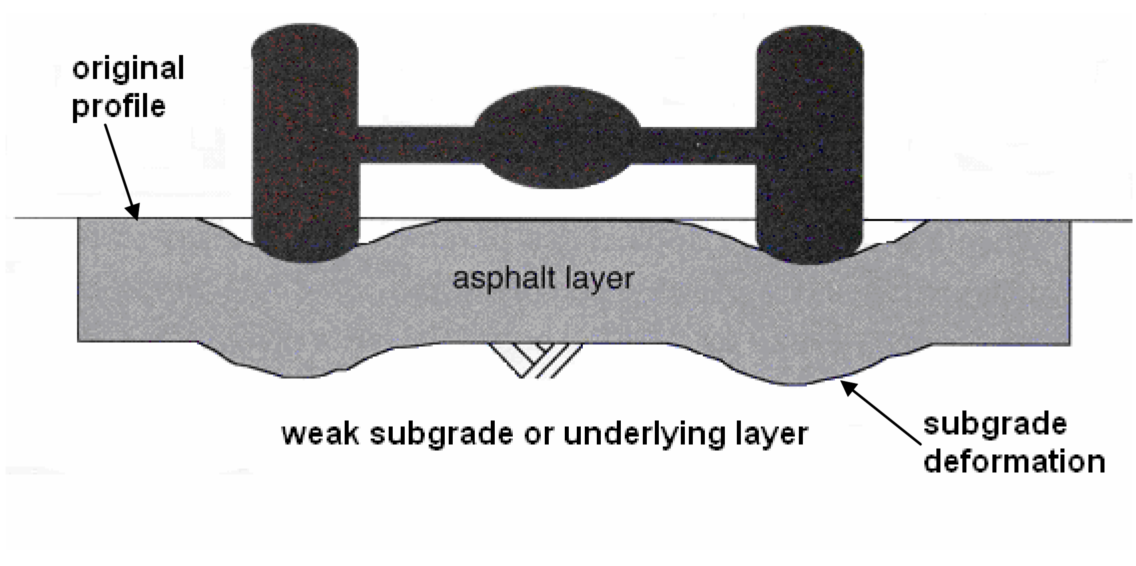

Figure 2.1 Rutting of Underlying Pavement Layers (after Asphalt Institute 1996)

Figure 2.2 Structural Rutting (courtesy Dr. C. F. Berthelot)

The second type of rutting is commonly known as visco-plastic rutting, or plastic flow rutting. This type of rutting is caused by deformation within the asphalt layers, and results from a lack of shear strength within the mixture to withstand repeated heavy loading. Multiple studies have identified this mechanism as a primary cause of rutting problems in North America (Huber and Heiman 1989, Sousa *et al.* 1991, Brown and Cross 1992). Prior to this research, SDHT has carried out other laboratory and field investigations to gain more insight into the plastic flow rutting mechanisms, as a result of continued problems with this type of rutting on Saskatchewan highways (Huber and Heiman 1986, Duczek 1987, Carlberg *et al.* 2002, Carlberg 2003, SDHT-2003-A).

Since hot mix asphalt concrete is a multi-phase particulate composite material that consists not only of asphalt cement and aggregate, but also air, the proper amount of asphalt for durability and the right balance between air voids and voids that are filled with asphalt are essential to achieve well-performing, rut resistant pavements (Roberts *et al.* 1996). Weak asphalt pavement can accumulate small, permanent strains under repeated load application, due to temperature effects on the mechanical behaviour of the asphalt mix. The strains can be a compilation of vertical consolidation as well as lateral shear, resulting in a depression caused by downward and lateral movement of the mixture, as illustrated in Figure 2.3. This is the primary reason why multi-axial testing is needed to properly characterize the mechanistic behaviour of asphalt mixes.

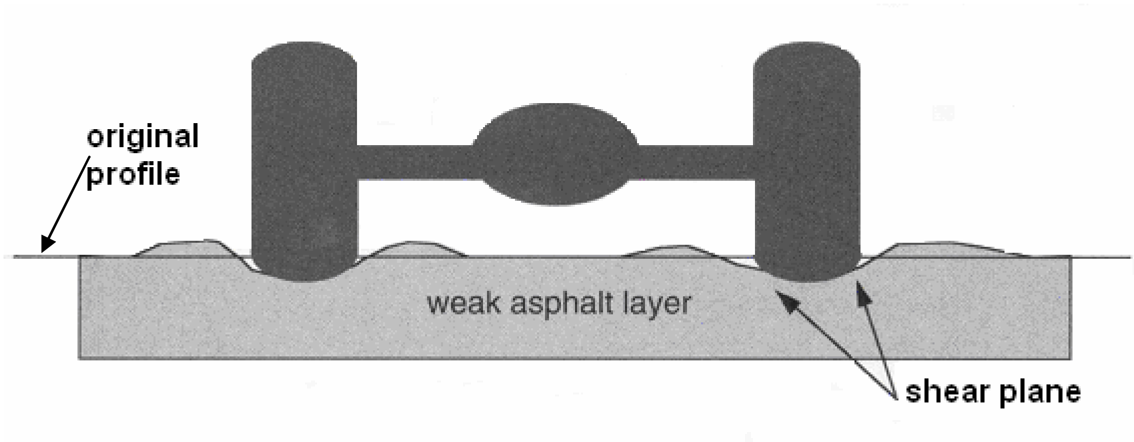

Figure 2.3 Rutting of Weak Asphalt Pavement Layer (after Asphalt Institute 1996)

When field temperatures are high, asphalt cement expands, due to its relatively high coefficient of thermal expansion, which increases with increasing temperature. Due to the considerable difference in the coefficients of thermal expansion for asphalt cement and aggregate (60 x 10^{-6}/°F for asphalt cement, and 3 to 6 x 10^{-6}/°F for aggregate), asphalt cement within HMAC will attempt to expand more than the aggregate skeleton, resulting in thermally induced stresses on the aggregate. When the air voids in the mix reach a low threshold, which is considered to be two to three percent for dense-graded mixes (Parker and Brown 1992), there is insufficient space to accommodate the expansion of the asphalt cement. The aggregate particles are pushed apart by asphalt cement, therefore losing interlock, and resulting in a weakened aggregate skeleton. Once the aggregate skeleton structure is compromised, additional loading results in the mixture being displaced outside of the rut, forming ridges on either side of the rut, similar to Figure 2.4. To compound the susceptibility to permanent deformation, asphalt cement viscosity decreases with increased temperature, making the asphalt concrete mix more vulnerable to deformation under load (Asphalt Institute 1996).

There are many factors that affect asphalt concrete mix susceptibility to permanent deformation (Huber and Heiman 1989, Sousa *et al*. 1991). In addition to the proportions of voids within the mix, the physical and mechanistic properties of the asphalt cement and aggregate used to engineer the asphalt concrete mix are also critical to its performance. Asphalt cement must be selected with consideration of field state conditions, with particular attention to local historic high temperatures and expected traffic loading. Given that aggregate is the main load carrying component of an asphalt concrete mix, especially at high temperatures, aggregate properties are critical in providing resistance to deformation under load (Field 1958, Davis 1995). It is commonly understood that larger, more angular aggregates, with rough surface texture increases rutting resistance (Brown and Bassett 1999, Button *et al.* 1990, Sousa *et al.* 1991, Kandhal and Mallick 2001).

Figure 2.4 Visco-plastic Rutting in Asphalt Pavement Layer (courtesy Dr. C .F. Berthelot)

2.3 Types of Hot Mix Asphalt Concrete Mixes

Flexible pavements distribute and transfer traffic loads to the prepared roadbed (subgrade), and consist of one or more lifts of HMAC and/or aggregate base and subbase placed above the prepared subgrade. In general, hot mix asphalt concrete can be defined as a particulate composite mixture of aggregate and asphalt cement. The term "hot mix" comes from the fact that the materials are heated during mixing, to remove any presence of moisture in the aggregate, to heat the aggregate, and to liquefy the asphalt cement for proper mixing and coating of aggregate (Asphalt Institute 1997). There are many different types of hot mix asphalt mixtures, serving a multitude of roles in road transportation applications. Typically, the different types of mixes are classified based on their aggregate gradation characteristics. This section covers only the common mix types, as they relate to this research, including dense-graded, open-graded, and gap-graded mixes. The grain size distributions of these mixes are illustrated in Figure 2.5.

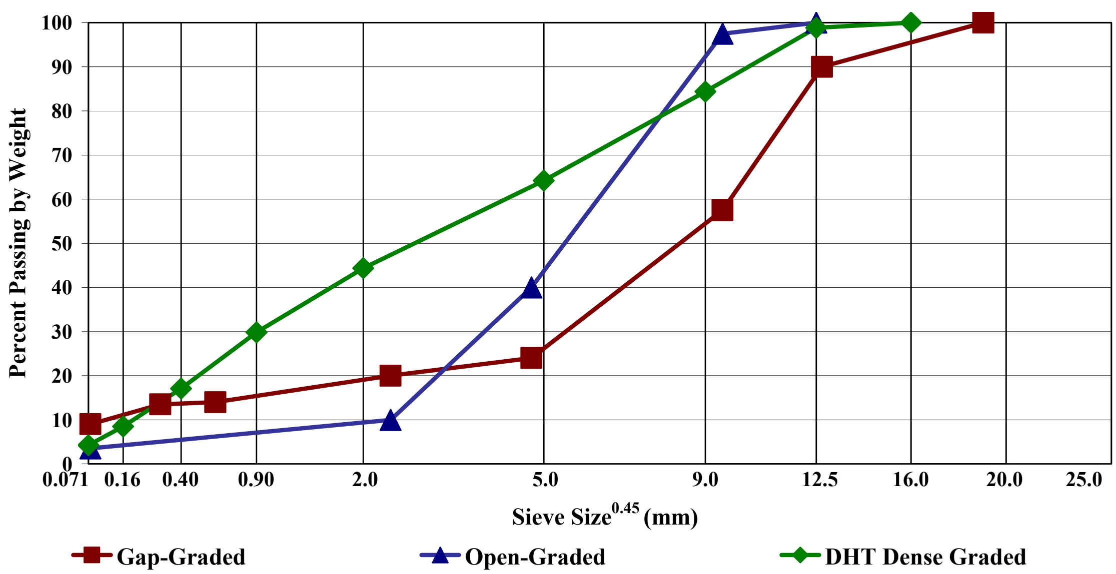

Figure 2.5 Aggregate Gradations of Common Asphalt Mix Types

2.3.1 Dense-Graded Mixes

Dense-graded asphalt mix relies on an aggregate skeleton that is well-graded, meaning that its gradation is relatively evenly distributed ranging from fine to coarse aggregate particles. Dense graded mixes are particularly useful in areas where hot mix aggregate is manufactured from glacial gravel deposits (Yoder and Witczak 1975), because gravel deposits tend to also be well graded. Although different types of mixes have been evaluated in the past (Siciliano and Qayyum 1994), Saskatchewan Highways and Transportation relies on dense graded mixes for all provincial hot mix asphalt concrete needs due to their economics and constructability.

An example of a SDHT dense-graded gradation is shown in Figure 2.5 and Figure 2.6. A surface photograph of a SDHT dense-graded mix is shown in Figure 2.7. Due to the uniform distribution of particle sizes and the gradation near maximum density, dense-graded mixes are relatively impermeable. Dense-graded mixes are versatile and can be used in all pavement layers, for all traffic conditions (NAPA 2001). They have been proven to work in a multitude of applications, including structural layers, surface friction courses, and levelling and patching, making it the most common asphalt mixes used today.

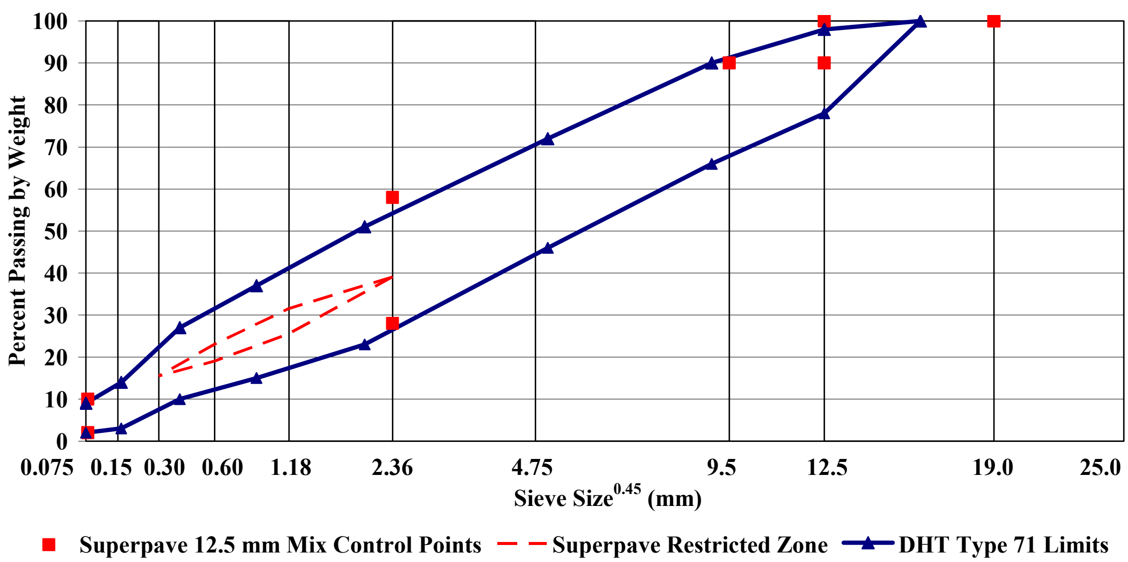

Figure 2.6 SDHT and Superpave™ Dense-Graded Gradation Comparison

The performance of dense-graded HMAC mixes was studied extensively under the Strategic Highway Research Program (SHRP) implemented by the US Congress in 1987 (Roberts *et al*. 1996). As a result of this research program, a new asphalt concrete mix design system called Superpave™ (Superior Performing Asphalt Pavements) was introduced. The Superpave™ mix design method specifies dense-graded mixes by implementing control points and a restricted zone on the standard gradation plot, to control the shape of the gradation curve, as illustrated in Figure 2.6 (Asphalt Institute 1996). In addition, Superpave™ encouraged coarse mixtures passing below the restricted zone, similar to the mix shown in Figure 2.7. Avoiding the restricted zone was incorporated to eliminate mixes that possess too much fine sand in relation to total amount of sand, which was known to result in compaction problems during construction, and increased susceptibility to permanent deformation (Asphalt Institute 1996). Recent research indicates that well-performing mixes can be achieved by going above and through the restricted zone, as well as below it, suggesting that the restricted zone could be eliminated altogether (Hand and Epps 2001, Kandhal and Cooley 2002).

SDHT Type 70 HMAC Mix **Superpave™ HMAC Mix**

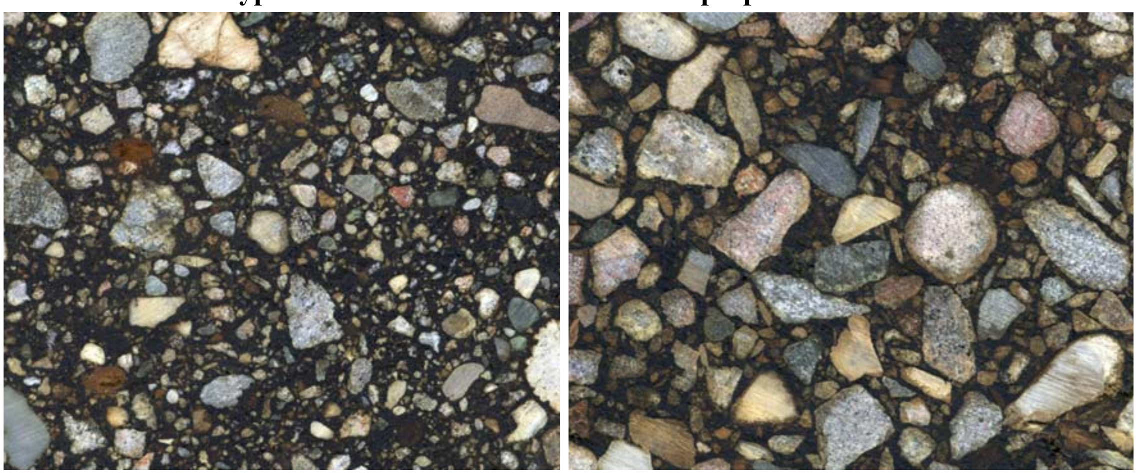

Figure 2.7 SDHT Type 70 Dense Graded HMAC Mix and Superpave™ Dense Graded HMAC Mix Cross Section

2.3.2 Open-Graded Mixes

Open-graded asphalt mixtures are porous mixes with interconnected voids, resulting in increased permeability. Their gradation is referred to as open, due to the mixtures have a larger portion of one-sized coarse aggregate, and only a small portion of fine aggregate, resulting in more air voids, since there is not enough fine aggregate to fill the spaces between the large rocks. As shown in Figure 2.5, the gradation line of an open-graded mix has a flat appearance in the small size range and is very low on the vertical scale, indicating a low amount of fine particles. The open-graded mixtures typically consist entirely of crushed stone, or in some cases they are made with crushed gravel with small amounts of manufactured sands (NAPA 2001). Open-graded mixes are used when a permeable asphalt mix layer is desired, and/or when increased traction in wet conditions is important.

The most common applications of the open-graded mixes in North America are the Open Graded Friction Courses (OGFC). The OGFC mixes are used as surface courses only, usually in areas of high rainfall and high traffic speeds, since their advantages include reduced tire splash in wet weather, good skid resistance, and tire-noise reduction. Modified asphalts and fibre additives are recommended with OGFC mixes to increase the asphalt cement content, to provide better durability and performance, and to prevent asphalt cement draindown during the curing period. In addition to issues with increased cost for premium crushed material and modified asphalt products, open-graded mixes may experience performance concerns related to clogging of the open pores with time, resulting in reduced drainage properties (Kuennen 2003-A). Winter maintenance is also a concern with OGFC mixes, since traditional applications of sand and salt blend would clog the mix.

2.3.3 Gap-Graded Mixes

Another type of HMAC is the gap-graded mix. Gap-graded aggregate is one that consists of only a small percentage of particles in the mid-size range of the gradation, resulting in the gradation curve being flat in the mid-size region, as illustrated in Figure 2.5. A popular gap-graded application is the stone matrix asphalt (SMA). The SMA mixture typically consists of high quality stone-on-stone skeleton, asphalt cement, manufactured sands, mineral filler and additives such as dust, polymers and/or fibres (NAPA 2001). The main benefit of SMA mixes is their resistance to permanent deformation, which is largely attributed to the use of cubical, angular crushed stone, and the stone-on-stone contact of the coarse aggregate. In SMA mixes, the coarse aggregate distributes and transfers the load to the underlying layers (Roberts *et al.* 1996). Although the capital costs to construct gap-graded mixes can be as high as 50 percent greater than those of conventional asphalt mixes, SMA mixes are known for their durability and high level of service, they are said to outperform Superpave™ mixes, and their service lives can be up to 20 to 30 percent longer than those of dense-graded HMAC (Kuennen 2003-B).

2.4 Physical Properties of Aggregate

Aggregates comprise 94 to 95 percent of an asphalt concrete mix by weight (Root 1989), and they are the main load carrying component. It, therefore, follows that selecting aggregates with desirable chemical and physical properties is an important step in achieving pavement quality and durability (Asphalt Institute 1998). Agencies usually rely on internally developed aggregate specifications, which are based on past experience. The researchers involved in the Strategic Highway Research Program (SHRP) adopted some of the commonly used material selection guidelines as part of the recently developed Superpave™ mix design method, by specifying "consensus" and "source" properties (Kennedy *et al.* 1994). Consensus properties are ones that pavement experts widely agreed on were critical to HMAC performance, and their critical values were also widely accepted. Those properties are coarse aggregate angularity, fine aggregate angularity, flat and elongated particles, and clay content. Source properties are ones that are also critical to HMAC performance, but a consensus could not be reached by experts as to what the specified values should be because they tend to be source-specific. Source properties that are listed by SHRP include toughness, soundness, and deleterious materials (Asphalt Institute 1996).

2.4.1 Aggregate Gradation and Top Size

Particle size and distribution are two of the most influential aggregate properties in hot mix asphalt concrete. Gradation characteristics influence the permanent deformation potential of a hot mix asphalt (Sanders and Dukatz 1992, Haddock *et al.* 1999, Kandhal and Mallick 2001). Gradation influences stability, permeability, durability, fatigue resistance, frictional resistance and resistance to moisture damage (Roberts *et al.* 1996). It is for that reason that gradation is one of the main properties included in most asphalt mix specifications and used to classify aggregates.

The American Society for Testing and Materials (ASTM) defines coarse aggregates as particles retained on a No. 4 (4.75 mm) sieve, and fine aggregate as that which passes the 4.75 mm sieve (Roberts *et al.* 1996). SDHT uses Canadian Metric Sieve Series for particle size and gradation determination, applying the 5 mm metric sieve to differentiate between the fine and coarse aggregate for laboratory testing purposes.

Along with gradation, the top size of the aggregate is also thought to be an important parameter, especially when considering the susceptibility to permanent deformation. Mixes with larger aggregate design are thought to be stronger than mixes prepared with smaller aggregate. (Brown and Bassett 1990, Kandhal and Mallick 2001). SDHT currently specifies three types of aggregate gradations for Hot Mix Asphalt Concrete mixes; Type 70, Type 71 and Type 72. The tolerance bands for each SDHT gradation type are listed in Table 2.1 and illustrated in Figure 2.8 (SDHT 2003-B).

The coarsest gradation used by SDHT is Type 70, with a top aggregate size of 18 mm, and a nominal maximum aggregate size of 16 mm. This type of gradation is thought to be well suited for high traffic loading roadways. Type 71 gradation has a top size of 16 mm, and a nominal maximum size of 12.5 mm, and is frequently used if the Type 70 gradation is not feasible to manufacture, or if the Type 72 gradation is thought to be too fine to withstand the predicted loadings over the life of the pavement. Type 72 gradation is the finest hot mix aggregate specified by SDHT, with a top size of 12.5 mm, and a nominal maximum size of 9 mm. Type 72 gradation is only used for asphalt mixes intended for top lifts. As can be seen, there is significant overlap between the three gradation bands.

SDHT selects hot mix aggregate gradations based on the maximum aggregate size that can be reasonably produced from the gravel source selected for the project, along with expected lift thickness. Quality of the gravel source and ability to manufacture the volume of aggregate required at an affordable cost also play a role in selecting HMAC aggregate type (SDHT 2001-B). Workability and ease of handling and placement have also recently become significant, due to difficulties with segregation during construction.

Table 2.1 SDHT Hot Mix Asphalt Aggregate Gradation Specification (SDHT 2003-B)

Sieve Size (mm)	Percent Passing by Weight					
	Type 70		Type 71		Type 72	
	Minimum	Maximum	Minimum	Maximum	Minimum	Maximum
18.0	100	100	100.0	100.0	100.0	100.0
16.0	78.0	98.0	100.0	100.0	100.0	100.0
12.5	68.0	92.0	78.0	98.0	100.0	100.0
9.0	54.0	80.0	66.0	90.0	66.0	90.0
5.0	38.0	65.0	46.0	72.0	46.0	72.0
2.0	18.0	46.0	23.0	51.0	23.0	51.0
0.90	10.0	33.0	15.0	37.0	15.0	37.0
0.40	5.0	25.0	10.0	27.0	10.0	27.0
0.16	3.0	13.0	3.0	14.0	3.0	14.0
0.071	2.0	9.0	2.0	9.0	2.0	9.0

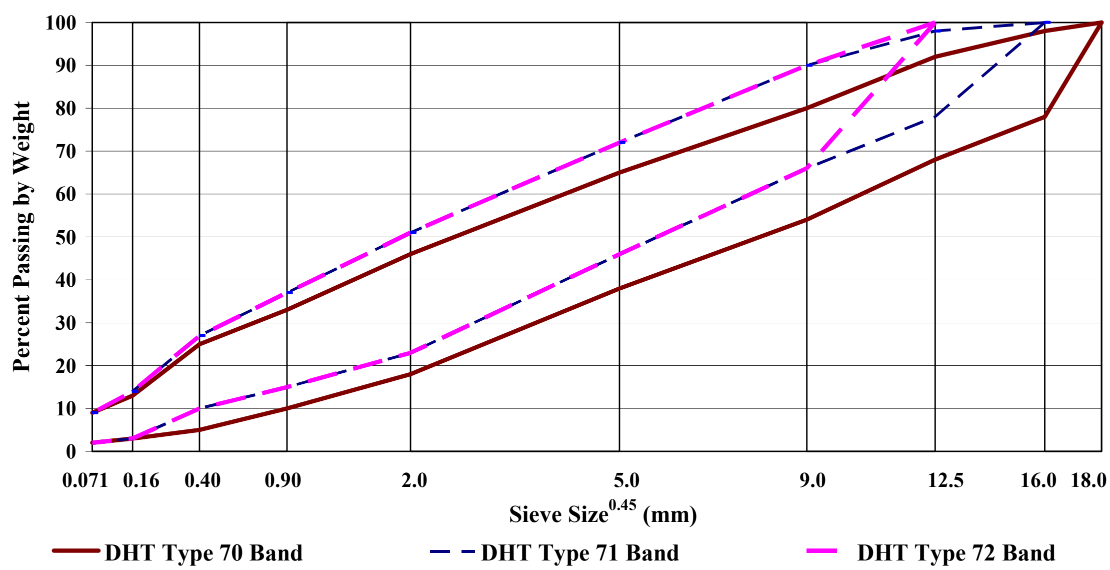

Figure 2.8 Aggregate Gradation Bands of SDHT Hot Mix Asphalt Concrete Mixes

2.4.2 Aggregate Shape, Angularity, and Texture

Particle shape, angularity and texture play an important role in bituminous mixes, influencing the load transfer capabilities of the aggregate structure (Field 1958, Ahlrich 1996). Shape and texture of the fine aggregate (smaller than 5 mm in diameter) are of particular importance in dense graded mixes, because the coarse aggregates (greater than 5 mm in diameter) are usually not in contact with each other; rather, they are suspended in the fine aggregate, which is forced to carry the load (Roberts *et al.* 1996). Figure 2.7 shows the arrangement of aggregate particles in a saw-cut sample from a SDHT mix with Type 70 gradation, illustrating that the large rocks are not necessarily in contact with each other in this particular mix.

Suitable HMAC aggregates are cubical rather than flat, thin, or elongated. Angular rather than rounded shape is also preferred. Angularity creates greater interlock and internal friction between particles, therefore resulting in greater mechanistic stability than can be achieved with rounded particles (Field 1958, Sousa *et al*. 1991, Tayebali 1998). Although mixes with rounded particles, such as natural sands and gravels, are more workable and compact easily; they are also more likely to continue compacting under traffic loading, resulting in rutting due to low air voids and plastic flow (Button *et al.* 1990, Emery 1990).

Surface texture of the aggregate also influences the strength and workability properties of HMAC (Ahlrich 1996). Rough-textured surfaces, such as those of crushed rock, result in stronger mixes by providing more friction between aggregate faces. Rough-textured aggregates typically result in higher voids in the compacted mixture, providing additional space for asphalt cement. The asphalt cement is thought to create stronger mechanical bonds with rough-textured aggregate, than with smooth aggregate (Roberts *et al*. 1996).

2.4.2.1 Coarse Fracture

In areas where glacial gravel deposits are used as HMAC aggregate sources, the only way of obtaining aggregate with angular particles, rough surface texture and improved distribution of particle size and range, is to manufacture it through mechanical crushing and sorting of the source gravel (Asphalt Institute 1983). The crushing process for SDHT HMAC aggregate typically involves screening off natural material smaller than 9 mm, and crushing the remaining aggregate. The resulting manufactured material is split on the 9 mm screen, into a manufactured fines and a manufactured coarse stockpile, respectively. Figure 2.9 illustrates the effect of mechanical crushing on aggregate particle shape of SDHT hot mix aggregate retained on the 5 mm sieve and passing the 9 mm sieve.

In an attempt to indirectly control shape, texture and angularity of the aggregates manufactured for use in HMAC production, many agencies specify a minimum amount of fracture necessary in coarse aggregate. Fracture is obtained by a visual count of coarse rocks that have mechanically fractured faces, and expressed as percent of the coarse portion of aggregate. SDHT defines fractured aggregate as that which has one or more mechanically fractured face (STP 206-14), while some agencies use a minimum of two faces.

The amount of coarse fracture required for SDHT mixes varies depending on the application of the HMAC, the asphalt cement used, and the type of aggregate gradation. SDHT employs the Marshall mix design method with 50 or 75 blows compactive effort. Fracture requirements are greater for pavements intended for high traffic loadings, which are designed with the 75 blow Marshall mix design.

When softer asphalt cement is selected to reduce cracking susceptibility, SDHT decreases the fracture requirement for economic reasons, since the softer asphalts are typically used on roads with lower traffic volumes. Aggregates with smaller top size are required to meet higher fracture expectations. Minimum requirements for fracture specified by SDHT are summarized in Table 2.2 (SDHT 2003-B).

It has been long realized that mechanically fractured coarse aggregate produces more stable mixtures (Field 1958, Wedding and Gaynor 1961, Emery 1990, Sousa *et al.* 1991). However, it has been suggested that increasing the content of crushed coarse aggregate past a certain amount (75-85 percent) in dense-graded mixes results in only marginal gains in mechanical stability and/or mechanical behaviour of the mix

Natural Aggregate **Mechanically Fractured Aggregate**

Figure 2.9 Effect of Mechanical Crushing on Aggregate Shape

(Wedding and Gaynor 1961, Carlberg 2003). Attempts to correlate coarse fracture to field performance have met with marginal success. Only when analysing field mixes with air voids above the minimum specified field voids for dense graded mixes were possible relationships visible (Huber and Heiman 1989, Brown and Cross 1992, Parker and Brown 1992, Carlberg 2003).

Table 2.2 SDHT Specifications for Coarse Fracture in HMAC Aggregate

	Gradation Type		
	Type 70	**Type 71**	**Type 72**
150/200A or 200/300A Asphalt Cement			
75-blow Marshall, Fracture, Minimum (%)	75	85	95
50-blow Marshall, Fracture, Minimum (%)	**60**	70	80
300/400A Asphalt Cement			
75-blow Marshall, Fracture, Minimum (%)	75	85	95
50-blow Marshall, Fracture, Minimum (%)	50	60	70

2.4.2.2 Fine Aggregate Angularity

The voids in a packed mass of angular, rough-textured aggregate are usually higher than those of an aggregate with smooth, rounded particles (Roberts *et al.* 1996). This concept has been applied in various test methods to describe the physical properties of aggregate. Recognizing the importance of the physical properties of fine aggregate in dense-graded mixes, SHRP researchers included a test method (ASTM C 1252) and specifications for fine aggregate angularity in the Superpave™ mix design (Asphalt Institute 1996).

Fine aggregate angularity (FAA) is defined as the percent of air voids present in a loosely compacted aggregate that passes the 2.36 mm sieve (Cominsky *et al.* 1994). In ASTM C 1252 fine aggregate of a prescribed gradation is poured into a cylinder with the use of a funnel. The filled cylinder is weighed, and the amount of voids in the sample is computed using the volume of the cylinder and the specific gravity of the dry aggregate. The Superpave™ criteria for fine aggregate angularity are shown in Table 2.3 (Asphalt Institute 1996). Although it is commonly understood that fine aggregate angularity is important, industry considers the current Superpave™ specifications of minimum 45 percent excessive, even for high traffic volumes (Huber *et al.* 1998).

Generally, angular and rough-textured aggregate will have a fine aggregate angularity greater than 45, whereas rounded, smooth-textured aggregates typically result in FAA values less than 43 (Ahlrich 1996). Fine Aggregate Angularity of 43 has been used in the past to divide acceptable and unacceptable mix performance in terms of permanent deformation (Brown and Cross 1992). Although the uncompacted void content test method has been proven to successfully differentiate between angular and rounded particles, and rank progressively angular gradations (Kandhal *et al.* 1991, Ahlirch 1996, Tayebali *et al.* 1998), research intended to relate fine aggregate angularity to performance has met with limited success. FAA values generally do not correlate well to rutting experienced in the field (Brown and Cross 1992, Parker and Brown 1992). This may be related to the fact that FAA only measures a small portion of the aggregate skeleton, namely that passing the 2.36 mm sieve. Attempts to correlate fine aggregate angularity to phenomenological tests such as Marshall stability and accelerated rut testers have shown mixed results (Huber *et al.* 1998, Prowell *et al.* 2005). Attempts to correlate fine aggregate angularity to mechanical tests have also shown mixed results (Ahlrich 1996).

Table 2.3 Superpave™ Fine Aggregate Angularity Criteria

Traffic (million ESAL)	Depth from Surface	
	< 100 mm	**> 100 mm**
< 0.3	-	-
< 1	40	-
< 3	40	40
< 10	45	40
< 30	45	40
< 100	45	45
> 100	45	45

As discussed in the previous section, SDHT does specify minimum fracture on the coarse aggregate retained on the 5 mm sieve (SDHT 2003-B), and increasing coarse fracture can result in a higher amount of manufactured fines through the crushing process. However, SDHT specifications do not directly address a minimum amount of manufactured fine aggregate or total manufactured aggregate to be used in the HMAC design and construction. In addition, the specifications currently do not address any physical properties directly related to particle shape, angularity, or texture in the fine portion of the aggregate.

2.4.3 Clay Content

Amount of plastic fines is limited to prevent aggregate particles from binding together during production, creating weak spots in the asphalt mix. Clay content is the proportion of clay sized material contained in the fine aggregate fraction. SDHT uses the Sand Equivalent test to measure the amount of clay-sized fines compared to sand particles in the fine aggregate portion of the gradation. Aggregate passing the 5 mm sieve is mixed and agitated in a flocculating solution in a graduated cylinder. Once settled, the heights of suspended clay-sized particles and sedimented sand are measured.

The sand equivalent is a ratio of the height of sand to the height of clay-sized material expressed as a percentage (ASTM C 131, STP 206-5). SDHT specifies a minimum sand equivalent of 45 percent for all hot mix asphalt concrete mixes (SDHT 2003-B).

2.4.4 Flat and Elongated Pieces

Flat and elongated particles are undesirable in HMAC aggregate structure because they tend to break during construction and under traffic (Asphalt Institute 1996). Currently SDHT does not employ any specifications that address the amount of flat and elongated particles; however, SDHT laboratory staff utilize the standard test procedure specified by ASTM whenever flat and elongated particles are of interest during the mix design stage. The ASTM test is performed on aggregate coarser than 4.75 mm; however, as previously mentioned, SDHT employs metric sieves; therefore the 5 mm sieve is used to separate the coarse aggregates from the fine for the purposes of laboratory testing. A calliper device is used to measure the ratio of the largest dimension of an aggregate particle to its smallest dimension (Asphalt Institute 1996, ASTM D 4791). A particle is considered flat and elongated if its maximum to minimum dimension ratio is greater than five. SHRP specifies a maximum content of flat and elongated particles of ten percent by mass of coarse aggregate (Asphalt Institute 1996).

2.4.5 Deleterious Materials

Aggregates being considered for use in hot mix asphalt concrete should be clean and free of undesirable materials, such as lightweight particles (wood, shale, coal, etc.), clay lumps, organics, and soft particles (STP 206-9 and STP 206-15). SDHT specifies a maximum of one percent of lightweight pieces allowable by weight of total aggregate. All other deleterious materials such as clay lumps, organics, and other soft particles are limited to a maximum of two percent (SDHT 2003-B).

2.4.6 Adhesion to Asphalt Cement

Adhesion of asphalt cement to the aggregate in hot mix asphalt concrete depends not only on the chemical properties of the asphalt cement, but also on those of the aggregate. Chemical aggregate properties depend on the origin and history of the aggregate and its source. For mix durability and long term performance, it is expected that the asphalt cement will bond with the aggregate surface, and that this bond will be durable enough to withstand intrusion of water, therefore resisting stripping of the asphalt film. Adhesion properties of aggregate depend on whether the aggregate has a greater affinity for asphalt cement or for water, and on the electric charges of the aggregate surface. For example, positively charged aggregates such as limestone and dolomite have a higher affinity for asphalt cement, while negatively charged siliceous aggregates prefer to bond with water, and therefore have a lower affinity for asphalt cement. Also, the asphalt cement is thought to create stronger mechanical bonds with rough-textured aggregate, than with smooth aggregate (Roberts *et al*. 1996).

Poor adhesion between asphalt cement and aggregate particles results in moisture susceptibility problems during pavement life, commonly referred to as stripping. Stripping is defined as the loss of bond between asphalt cement and aggregate surface, resulting in exposed aggregate surface with minimal or no asphalt coating (STP 204-15). SDHT tests the potential for stripping in all aggregate for use in hot mix asphalt, since stripping results in mix durability problems and, therefore, shortens pavement life (Kennedy 1983). SDHT implements the Indirect Tensile Strength test procedure, which is based on the modified Lottman test (AASHTO T 283), to determine the loss of mechanical strength as measured by tensile strength in Marshall specimens water-cured at 60°C, compared to samples air-cured at 25°C, as shown in Equation 2.1. SDHT specifies a minimum of 70 percent of Retained Tensile Strength in the water-cured samples (SDHT 2003-B).

$$\%\ retained\ tensile\ strength = \frac{tensile\ stength\ (water\ cured\ at\ 60°C)}{tensile\ strength\ (air\ cured\ at\ 25°C)} \times 100 \qquad \textbf{(2.1)}$$

Once the indirect tensile strength test is complete, samples are physically split and visual stripping inspection is performed (STP 204-15). When broken, a sample of stripped asphalt concrete will appear brown due to moisture damage, with visible uncoated aggregate faces (as shown in Photo A in Figure 2.10), when compared to a well-coated mixture, with black asphalt cement and no exposed aggregate (as can be seen in Photo B in Figure 2.10). Anti-stripping additives are available to modify the chemical properties of aggregate surface, to facilitate bonding to asphalt cement.

Photo A: No Anti-Stripping Additives **Photo B: With One Percent Lime**

Figure 2.10 Stripping in Asphalt Concrete

SDHT mitigates aggregate stripping susceptibility by adding one percent of hydrated lime by weight of aggregate as an anti-stripping agent. When the lime cannot be accommodated in the asphalt aggregate gradation, liquid anti-stripping additives are used to provide improved bonding between the asphalt cement and the aggregate particles. SDHT preference for using hydrated lime stems from the fact that in addition to reducing moisture susceptibility, lime also improves field performance of SDHT mixes by reducing susceptibility to permanent deformation, oxidation, and fatigue cracking, and therefore lowering maintenance costs of lime-treated asphalt mixes (Beshara 2004).

In addition, hydrated lime affects the mechanistic properties of SDHT mixes by increasing the dynamic modulus and reducing axial micro strains when tested using rapid triaxial frequency sweep equipment characterization (Berthelot *et al.* 2005).

2.5 Physical Properties of Hot Mix Asphalt Concrete Mixes

Hot mix asphalt concrete consists of three components: mineral aggregate, asphalt cement binder, and air voids, as shown in a saw-cut cross-section of an HMAC pavement core in Figure 2.11. The relative proportions of these material components have long been recognized as significant in terms of HMAC field performance (Coree 1999).

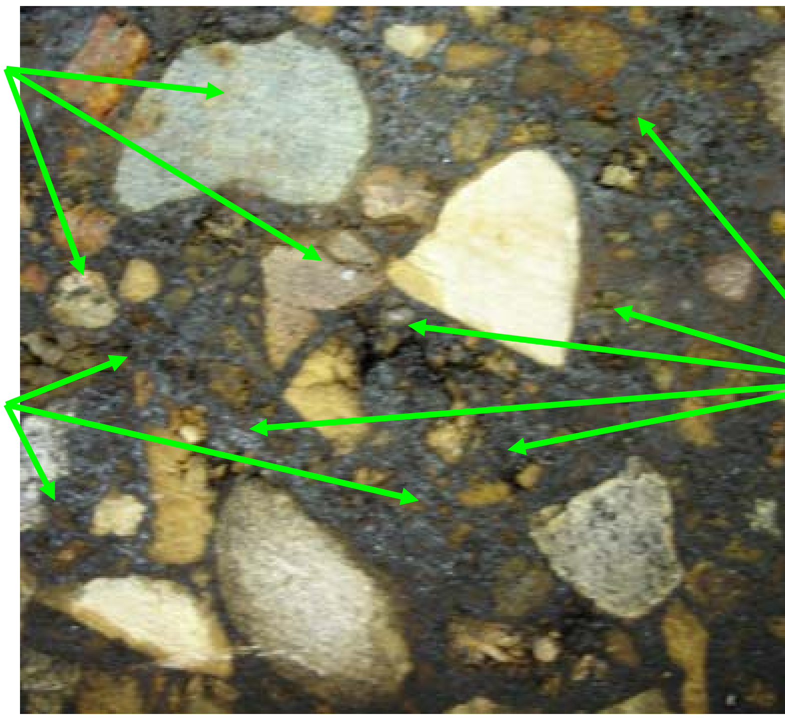

Figure 2.11 Saw-Cut Asphalt Concrete Surface

Although the acceptable ranges and limits may vary, the important physical volumetric parameters, such as the amount of air voids and the level of air voids filled with asphalt, form the basis of current asphalt concrete mix design methods (Foster 1993, Asphalt Institute 1996). This section briefly defines the common physical mix properties measured, with reference to the current SDHT specifications for hot mix asphalt concrete.

1.1.1 Voids in the Mineral Aggregate

Voids in the Mineral Aggregate, or VMA, is the total volume of void space available in a compacted aggregate structure. VMA governs the minimum asphalt content and affects the long term performance of HMAC pavements, and has been widely accepted as one of the main control parameters for the design of dense-graded mixes (Aschenbrener and MacKean 1994, Kandhal and Chakraborty 1996). It can be calculated using bulk specific gravity of aggregate (Roberts *et al.* 1996):

$$VMA = 100 - \frac{M_{Agg} / G_{sb}}{M_T / G_{mb}} * 100 \qquad \textbf{(2.2)}$$

where:

VMA = Voids in mineral aggregate

M_{agg} = Mass of aggregate

G_{sb} = Bulk specific gravity of aggregate

M_T = Total mass of mixture

G_{mb} = Bulk specific gravity of compacted mixture

Appropriate VMA is required to provide space in the mix for enough asphalt cement to achieve proper aggregate coating and bonding, as well as to leave air voids for the thermal expansion of asphalt cement during high in-service temperatures. VMA in a compacted asphalt concrete has two components: the volume of voids that is filled with asphalt cement, and remaining voids filled with air (Roberts *et al.* 1996). SDHT specifies a range of acceptable VMA depending on the gradation of HMAC aggregate used, as shown in Table 2.4 (SDHT 2003-B).

Table 2.4 SDHT Specifications for Voids in Mineral Aggregate

Gradation Type	VMA (%)	
	Minimum	Maximum
Type 70	13.5	15.5
Type 71	14.0	16.0
Type 72	14.0	16.0

2.5.1 Voids in Total Mix

Sufficient air voids in a compacted asphalt mix are required to allow for thermal expansion of asphalt cement at high temperatures. Voids in Total Mix (VTM) are a measure of the volume of voids remaining in the mix after compaction, and is expressed as (Roberts *et al.* 1996):

$$VTM = \left[1 - \frac{M_T / V_T}{M_T / (V_{EAC} + V_{Agg})}\right] * 100 \qquad \textbf{(2.3)}$$

where:

VTM = Voids in total mix

M_T = Total mass of compacted specimen

V_T = Total volume of compacted specimen

V_{EAC} = Volume of effective asphalt content

V_{Agg} = Volume of aggregate (bulk)

Numerous studies have linked insufficient air voids to loss of strength during hot weather, resulting in permanent deformation (Brown and Cross 1992, Emery 1990, SDHT 2003-A). In response to significant rutting problems on selected provincial highways, SDHT conducted a major rutting study in 1986. The study concluded that rutting correlated with asphalt content, VTM, and voids filled with asphalt, although the correlation coefficients obtained were marginally acceptable (Huber and Heiman 1986). Another SDHT internal study in 1987 examined behaviour of full depth asphalt mix and found that low air voids were one of the primary contributing factors to plastic flow deformation (Duczek 1987).

During the mix design stage, engineers aim to simulate an air void content in the laboratory compacted mix representative of that in a field mix after several years of service. Similar to commonly used specifications for dense-graded mixes (Asphalt Institute 1997), SDHT specifies a range of acceptable VTM from three to five percent.

2.5.2 Voids Filled with Asphalt

The part of VMA that is occupied by effective asphalt cement (that which is not absorbed by the aggregate itself) is referred to as Voids Filled with Asphalt, or VFA. The amount of voids filled with asphalt is directly related to the amount of void space available in the aggregate skeleton (VMA) and to the amount of air voids (VTM); therefore, it is an important parameter that is found to relate to asphalt pavement performance (Huber and Heiman 1986). During an in-service rutting investigation of select Saskatchewan asphalt pavements, it was found that VFA influences the amount of rutting observed during the service life of an asphalt pavement (Carlberg *et al.* 2002). SDHT specifies a range of acceptable VFA of 65 to 78 percent.

$$VFA = \frac{VMA - VTM}{VMA} * 100 \qquad \textbf{(2.4)}$$

where:

VFA = Voids filled with asphalt

VMA = Voids in mineral aggregate

VTM = Voids in total mix

2.5.3 Asphalt Film Thickness

Thin asphalt coating is one of the parameters linked to excessive aging of asphalt binder, resulting in decreased pavement life. Asphalt film thickness is a calculated parameter, which is determined by dividing the total surface area of the aggregate obtained from its gradation, by the effective asphalt content:

$$T_f = \frac{V_{asphalt}}{SA * M} * 1{,}000 \qquad \textbf{(2.5)}$$

where:

T_f = Average film thickness (microns)

$V_{asphalt}$ = Effective volume of asphalt cement (litres)

SA = Surface area of the aggregate (m^2 per kg of aggregate)

M = Mass of aggregate (kg)

The surface area of particles is calculated based on the gradation of the aggregate being used in the hot mix asphalt. Total percent passing each sieve size is multiplied by a surface area factor, and the sum of these products represents the surface area of the sample in m^2/kg. The surface area factors are provided by the Asphalt Institute (Asphalt Institute 1997), with a caution that they assume spherical shape of aggregate, and are intended as an index factor only. The factors were developed for Imperial sieve sizes; therefore, SDHT converts the amount passing the SDHT standard metric sieve sizes to what would pass the Imperial sieves, and then the surface area factors shown in Table 2.5 are applied, and an estimated film thickness is calculated.

Table 2.5 Surface Area Parameters for Asphalt Film Thickness Calculations

Imperial Sieve Number	Surface Area Factor (m^2/kg)
#4	0.0041
#8	0.0082
#16	0.0164
#30	0.0287
#50	0.0614
#100	0.1229
#200	0.3277

Although film thickness is a purely conceptual parameter, and is based on many assumptions, it is a common approach used by design engineers to quantify the coating of the aggregate particles in the asphalt concrete mixture (Kandhal and Chakraborty 1996). Ensuring an adequate film thickness protects against premature pavement cracking caused by oxidation, because if the asphalt cement is too thin, air can more readily oxidize the thin films resulting in brittleness (Roberts *et al.* 1996). SDHT specifies a minimum film thickness of 7.5 μm.

2.6 Asphalt Concrete Mix Design Methods

Asphalt concrete mix design methods attempt to balance the composition of aggregate and asphalt cement to achieve long lasting performance in a pavement structure. Laboratory testing is conducted to determine the optimum proportion of the materials to achieve:

- Minimum sufficient asphalt cement content to coat the aggregate to ensure durability and to maximize cost effectiveness of the amount of asphalt cement added.
- Adequate mix stability to withstand the traffic conditions without distortion.
- Sufficient air voids in the compacted mix to accommodate a small amount of compaction under traffic, and to allow for asphalt expansion during high temperatures without compromise of performance.
- An asphalt concrete mix that is relatively impermeable, to limit the intrusion of air and moisture which may affect durability.
- Workability that allows for efficient placement of the mix during construction, without segregation, and without compromising performance.
- Sufficient skid resistance in inclement weather (Asphalt Institute 1997).

Besides selecting suitable aggregates and asphalt cement type, the traditional mix design process involves preparing and compacting laboratory samples of trial mixes, determining their volumetric properties, assessing stability through mechanical testing, and analysing results to determine the most suitable mixture composition for the specified conditions.

Traditional mix design methods are based on phenomenological-empirical concepts that do not measure fundamental mechanistic material properties, and therefore do not relate directly to field performance. The specifications for the design parameters used by traditional methods have been empirically developed by correlating the laboratory test results of phenomenological tests with the performance of the paving mixes in the field. Since these correlations were made for specific conditions, their application is limited to those specific conditions (Cominsky 1994, Roberts *et al*. 1996, Asphalt Institute 1997). A brief summary of the common mix design methods is presented in the following sections.

2.6.1 Marshall Mix Design

The Marshall method of asphalt mix design was created by Bruce Marshall in the 1930's, for use by the Mississippi State Highway Department. The method was studied and modified in the following years by the US Army Corps of Engineers for use in designing asphalt pavement for aircraft (Foster 1993). The primary goal of the Corps of Engineers was to develop a quick, portable laboratory procedure that helped select proper asphalt cement content. Since then, this method has been standardized (AASHTO T 245, ASTM D 1559) and is widely used by many road agencies in North America as the primary method of asphalt pavement design (Hafez and Witczak 1995). Saskatchewan Highways and Transportation relies exclusively on the Marshall method for the design of HMAC mixes.

Once the aggregate proportions and the asphalt cement grade are selected, trial samples are compacted in the lab at various asphalt cement contents above and below the expected optimum. Once trial mixes are prepared, a Marshall hammer is used to compact laboratory specimens of 102 mm (4 inch) diameter with a height of 64 mm (2.5 in.). The Marshall hammer achieves compaction in a sample by dropping a 10-lb (4536-g) flat-faced weight onto the surface of the sample from a height of 18 inches (457.2 mm). The sample receives an equal number blows on each face (ASTM D 1559). The traditional Marshall hammer is a hand-held device, requiring the weight to be manually lifted and dropped by the operator to apply each blow. SDHT uses mechanical Marshall compactors such as the one shown in Figure 2.12. The SDHT compactors have a rotating base that moves between blows, and a bevelled hammer head. These compactors are correlated to the traditional hand-held, flat-faced hammer compactors, to determine the equivalent number of blows that need to be applied to the asphalt samples.

The level of compaction varies depending on expected traffic loading. Typically, for light traffic, 35 blows are used. For medium traffic, the 50 blow design is implemented, and mixes intended for roads with high traffic loadings are designed using 75 blows. SDHT uses the 50 blow design for roads with a design traffic loading up to three million ESALs. The 75 blow design is used for mixes intended for roads with traffic volumes higher than three million ESALs (SDHT 2001-B, SDHT 2003-B), which includes 3847 lane km of roads on the National Highway System (Frass 2007), and selected primary economic routes which carry high volumes of truck traffic.

To test the trial mixes for mechanical strength, the Marshall mix design utilizes the Marshall stability and flow apparatus, also known as the Marshall stabilometer, which is illustrated in Figure 2.13.

Figure 2.12 Marshall Compaction Apparatus at SDHT Laboratory

Figure 2.13 Marshall Stabilometer at the SDHT Laboratory

Samples are placed in a Marshall breaking head, which has an upper and a lower cylindrical segment with an inside width of two inches (50.8 mm), conforming to the diameter of the compacted asphalt concrete samples. The samples are heated to 60°C and placed in the assembly on their side. A vertical load is applied to the assembly at a rate of 2 inches/minute (50.8 mm/minute), until maximum load is reached. When the load begins to decrease, the test is stopped and the stability (maximum load) is recorded in pounds (Newtons). During the loading an attached dial gauge measures the specimen's plastic flow as a result of the loading. The flow value in 0.01 inch (0.25 mm) increments is recorded at the time when the maximum load is reached. This concept is illustrated in Figure 2.14. SDHT specifications for Marshall stability and flow are shown in Table 2.6.

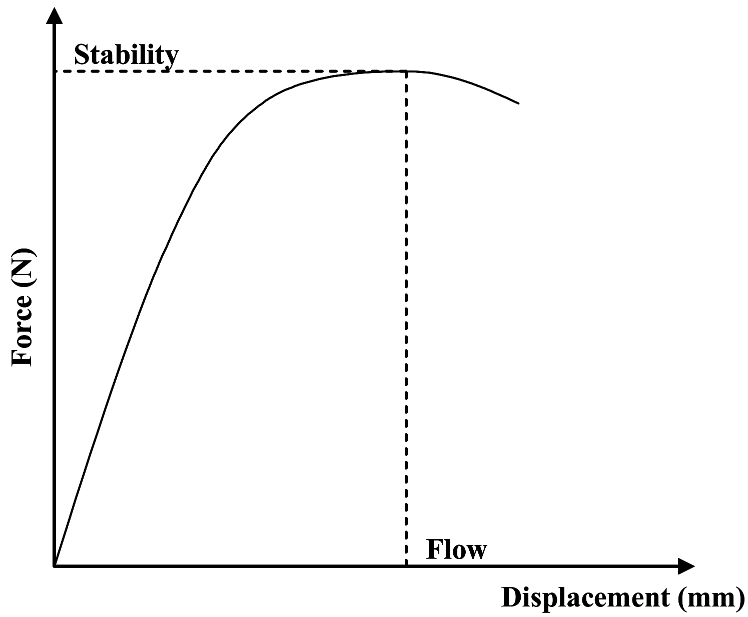

Figure 2.14 Marshall Stability and Flow Measurement

Table 2.6 SDHT Specifications for Marshall Stability and Flow (SDHT 2003-B)

Marshall Property	Level of Marshall Compaction	
	50 blows	75 blows
Stability, minimum (Newton)	5,500	7,000
Flow, range (mm)	1.5-3.5	1.5-3.5

The optimum asphalt content according to the Marshall mix design is chosen based on examining volumetric properties of the specimens as well as their stability and flow test results. Graphical plots are constructed for each parameter to observe changes with varying asphalt content. Typically, asphalt mix designers aim for an air void content between three and five percent (Asphalt Institute 1997). Agencies utilize specifications for volumetric properties, stability and flow, and the successful mix will meet all the requirements; otherwise, the aggregate gradation composition has to be modified and re-evaluated.

Although the Marshall method is very popular, there are several limitations associated with its ability to reliably predict performance. The Marshall hammer uses direct impact compaction, which does not simulate field compaction conditions, given that the asphalt rollers create more of a kneading type action than direct impact. Also, the compacted Marshall sample is very small in diameter, and studies have shown that four inch diameter samples have higher variability of stability and flow results than six inch samples, especially for mixes with large top aggregate size (Kandhal and Brown 1990, Lim *et al.* 1995). In addition, Marshall stability and flow results are different when sample diameter is changed (Kandhal *et al*. 1996). The question of scale in asphalt mix testing is an important one. Due to the fact that asphalt mix is a particulate composite material, in order to satisfy the concept of homogeneity of a sample, the sample should be large enough so that its global characteristics remain constant regardless of its location (Weissman *et al.* 1999). The Marshall mix design method utilizes a phenomenological-empirical approach to characterize asphalt concrete. The stability and flow parameters do not measure fundamental mechanistic material properties; therefore, they are not directly related to field performance, making their validity based solely on past experience with correlation to field performance.

2.6.2 Hveem Mix Design

The Hveem mix design method was developed over several years by Francis Hveem, a California materials engineer, and finalized in 1959 (Roberts *et al.* 1996). This mix design method was adopted by several state highway agencies in the United States, and continues to be used as a mix design method primarily in the Western United States (Linden *et al.* 1989, ASTM D 1560, AASHTO T 246).

A kneading compactor is used to compact samples of a diameter of 102 mm and a height of 63.5 mm. The Hveem compactor applies pressure to the asphalt mix in the mould through a hydraulic-powered tamping foot, while the base rotates between pressure applications. This kneading action is intended to simulate the rolling effect of pavement compaction equipment. An illustration of this apparatus is shown in Figure 2.15.

Figure 2.15 Hveem Kneading Compactor Foot and Rotating Base

Realizing that rutting was a major distress of asphalt mixes, and that there was a need to assess the asphalt mix and its ability to resist shear forces applied by wheel loads, Hveem developed the Hveem stabilometer, shown in Figure 2.16. In this apparatus, the compacted asphalt mix specimen is subject to a vertical load applied on the flat surface, and the amount of load transmitted horizontally is recorded. The perimeter of the specimen is confined in a diaphragm, and is surrounded by an oil reservoir, to simulate field loading conditions (Roberts *et al.* 1996). The increase of pressure in the oil is recorded as the horizontal pressure resulting from the vertical load.

Once the samples have undergone stability testing, the method originally included testing with the Hveem cohesiometer. This test equipment and method was developed to quantify cohesive strength across the diameter of a sample, and consists of bending the sample as a cantilevered beam until it fails. Although it was useful for oil mixes, HMAs tend to have large cohesion values as measured by the cohesiometer and rarely, if ever, fail. As a result, the cohesiometer is rarely used (Roberts *et al.*, 1996).

Based on the philosophy that hot mix asphalt requires sufficient stability to resist traffic loading, and that climatic durability increases with thicker asphalt films, the

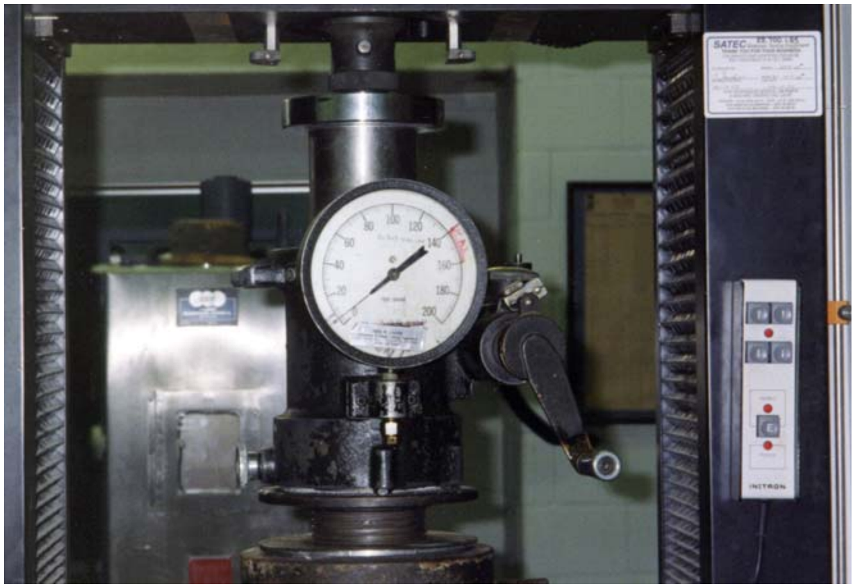

Figure 2.16 Hveem Stabilometer (Courtesy Dr. C.F. Berthelot)

design asphalt content is selected as that asphalt content resulting in the highest durability without dropping below a minimum allowable stability. Therefore, the asphalt content is maximized, while meeting minimum stability requirements.

When compared to the Marshall mix design, the Hveem mix design method has several advantages. The kneading compactor better simulates field compaction than does the direct impact compaction of the Marshall hammer. Also, Francis Hveem recognized the importance of quantifying stress and strain, and the need for creating equipment capable of testing both bound and unbound road materials. Thus test procedures were developed utilizing the stabilometer, not only for asphalt mix testing, but also to characterize subgrade strength, through measuring the R-value (ASTM D 2844). The Hveem mix design procedure incorporates important engineering principles; however, it also has several limitations. The specimen dimensions are limiting, for similar reasons as the Marshall method. Although the Hveem stabilometer measures multiaxial behaviour of asphalt concrete mixes, the test procedure was developed based on correlations between laboratory and field observations, and does not directly measure mechanistic material properties, which are necessary for mechanistic road modelling and performance prediction (Berthelot *et al.* 1999).

2.6.3 Superpave™ Level I Mix Design

The SHRP research program was initiated with one of the primary goals being to improve upon the shortcomings of the traditional mix design methods, and to evelop a new mechanistic, performance based hot mix asphalt concrete design procedure. As a result of this 1987 multi-million dollar undertaking by the USA and thirteen other countries, the Superpave™ asphalt concrete mix design method was created (Asphalt Institute 1996). The method consists of three levels of design, which are progressively more rigorous and involved. This method of design is becoming popular as a replacement for the traditional Marshall and Hveem methods. Many US and Canadian agencies have either already implemented, or are considering implementing the Superpave™ Level I method of mix design in part or in its entirety (Better Roads, January 2000).

Superpave™ is the first mix design system to employ mechanistic-based asphalt binder specifications, and mechanistic-related aggregate specifications. Traditionally, it was up to each agency to develop a rationale for aggregate and asphalt binder selection. This method also incorporates sophisticated compaction equipment thought to be more simulative of field compaction than the impact hammers used by the Marshall mix design method.

Under the Superpave™ Level I mix design, once the aggregates and asphalt cement are selected based on Superpave™ specifications, trial mixes are manufactured at various asphalt contents above and below the expected optimum, similar to the Marshall method. However, unlike the Marshall method, the specimens required for a Superpave™ mix design are considerably larger, with a diameter of 150 mm and a height of 150 mm. The larger samples allow for testing of mixes with larger top size aggregates, and provide better representation of mix behaviour by minimizing the influences of sample shape and variability. While the Marshall method attempts to select an asphalt content to satisfy a range of air voids of three to five percent, the optimum asphalt content in the Superpave™ method is selected based on a desired level of four percent air voids in the mixture compacted with the amount of compaction equivalent to what is expected during the design life of the pavement.

The most significant breakthrough in this asphalt concrete design method is no doubt the adoption of sophisticated compaction equipment, in the form of a Superpave™ gyratory compactor, shown in Figure 2.17. After reviewing gyratory compaction procedures which have been utilized around the world since the 1930's, the SHRP researchers modelled the Superpave™ gyratory compactor after the Texas gyratory compactor, and the French gyratory compactor used by Laboratoire Central des Ponts et Chaussees (LCPC) (Huber 1996).

Figure 2.17 SHRP Gyratory Compactor with Compacted Sample

The Superpave™ gyratory compactor employs a vertical pressure of 600 kPa, which is applied on a heated asphalt mix sample contained in a cylindrical mould with an inside diameter of 150 mm, while the mould itself is being gyrated around an angle of 1.25 degrees from a vertical axis. This computerized apparatus can monitor the increase in specimen density with increased compactive effort in real time. The densification is measured as a percent of the theoretical maximum specific gravity (G_{mm}) of the asphalt concrete mix being compacted.

The design level of compaction (N_{des}, or N_{design}) expressed by the number of gyrations of the compactor, is the amount of compaction expected in the field after the mix is subjected to the design number of ESALs. Therefore, the appropriate N_{des} is selected based on the design traffic volumes, and the local temperature regime (Cominsky *et.al.* 1994). Table 2.7 illustrates the selection criteria for compactive effort required. The Superpave™ mix design method utilizes two other critical threshold levels of compaction to control mix densification.

The initial number of gyrations (N_{ini}, or $N_{initial}$) is used to assess the compactability of mixes. If the mix densifies too quickly, problems with field densities and permanent deformation may be encountered. Superpave™ specifies a maximum allowable mixture density of 89 percent of G_{mm} at N_{ini}. The maximum number of gyrations (N_{max}, or $N_{maximum}$) represents a level of compaction that should in theory never be exceeded during the life of the pavement. The level of compaction in the asphalt mix should not exceed 98 percent of G_{mm} at N_{max} (Asphalt Institute 1996).

Table 2.7 Superpave™ Design Gyratory Compaction Effort (after Asphalt Institute 1996)

Design ESALs (million)	Average Design High Air Temperature											
	< 39°C			39 - 40°C			41 - 42°C			43 - 44°C		
	N_{ini}	N_{des}	N_{max}	N_{ini}	N_{des}	N_{max}	N_{ini}	N_{des}	N_{max}	N_{ini}	N_{des}	N_{max}
< 0.3	7	68	104	7	74	114	7	78	121	7	82	127
0.3 - 1	7	76	117	7	83	129	7	88	138	8	93	146
1 - 3	7	86	134	8	95	150	8	100	158	8	105	167
3 - 10	8	96	152	8	106	169	8	113	181	9	119	192
10 - 30	8	109	174	9	121	195	9	128	208	9	135	220
30-100	9	126	204	9	139	228	9	146	240	10	153	253
> 100	9	143	235	10	158	262	10	165	275	10	172	288

Although the Superpave™ Level I mix design system is a significant step forward in the state-of-the-art of pavement design, it does not incorporate mechanistic testing at temperatures representative to those in the field. Instead, the method uses volumetrics and mix response to compaction to indicate a suitable mix (Sousa *et al.* 1995), by specifying the number of gyrations for compaction depending on traffic loading, and by limiting the amount of densification at N_{ini} and N_{max}. There is a general feeling by the industry that physical mix properties alone are insufficient to select appropriate asphalt concrete mixes, and research is currently ongoing to develop a simple performance test that can be incorporated into the Level I mix design (Witczak *et al.* 2002).

2.7 Mechanistic Hot Mix Asphalt Concrete Material Characterization

The intent of any asphalt concrete mix design is to create a mixture that will withstand the loading and environmental conditions to which the pavement is subjected in the field. In addition to the physical characteristics of the mix, engineers focus on predicting the performance of the mixture under the field state conditions. Material characterization is the measurement and analysis of the response of HMA mixes to load, deformation, and/or the environment at various rates of loading and temperatures (Roberts *et al.* 1996). Traditional asphalt concrete mix design methods employ phenomenological-empirical materials tests, such as Marshall stability and flow, which are based on their correlation with field performance. These methods use simulative tests and experience-based knowledge of material behaviour in the field. The applicability of such tests is limited to the specific conditions upon which they were developed, and cannot be reliably adapted outside those parameters. These tests cannot be relied on to correctly rank mixes with respect to their permanent deformation performance (Brown *et al.* 2004).

In order to be able to predict material behaviour, the concepts of continuum mechanics and measures of fundamental material properties such as stress and strain need to be employed. While pavement engineering has traditionally relied on empirical-based materials testing, other engineering disciplines such as aerospace and mechanical engineering have successfully incorporated fundamental material properties for material characterization and mechanistic structural modeling (Allen and Haisler 1985). In fact, they have taken the next step, and are now researching the mechanics of critical failure conditions on a micro-scale (i.e. micro-damage mechanics). They are able to do so by having the necessary mechanistic material properties obtained from applying continuum mechanics (Goyal and Johnson 2003, McBagonluri *et al.* 2005).

Although it is more challenging to develop and implement tests that quantify fundamental paving material properties, the pavement engineering community has been interested in mechanistic testing for over fifty years (Yoder and Witczak 1975). The advantage of mechanistic-based tests is that they quantify fundamental thermo-mechanical material behaviour across various field state conditions, such as various stress and strain states, and temperatures. Because the properties measured by these tests are fundamental, apply to all materials, and do not change with time, they are the best choice for the basis of any performance prediction models.

2.7.1 Superpave™ Level II and III Mix Design

The SHRP research program invested 50 million (USD) into developing new, mechanistic-based performance-prediction test methods, some of which were incorporated into Superpave™ Levels II and III designs (Kennedy *et al.* 1994). The Superpave™ Shear Tester (SST) was implemented in Levels II and III to predict the development of permanent deformation and fatigue cracking in the mix over time. This sophisticated testing equipment is designed to simulate the high shear stresses that exist near the pavement surface at the edges of the vehicle tires (Cominsky 1994). However, the SST equipment is extremely expensive, the tests are complex to perform, and specimens need to be cut and glued before testing (Berthelot 1999, Brown *et al.* 2004). In a comprehensive study of various mixes at the Saskatchewan SPS-9A site, the Superpave™ Shear Tester was found to have a high coefficient of variation (CV) when compared with the variability of other test methods (Berthelot 1999).

SHRP made considerable progress in developing the theoretical material science to mechanistically characterize asphalt mixes and predicting performance (SHRP 1993). However, the Level II and Level III test methods although based on mechanistic principles, are complex, expensive, and time consuming. Also, recent research indicates that these tests may not be reliable for performance prediction (Anderson *et al.* 1999, Berthelot *et al.* 1999). For these reasons, the Level II and Level III tests are rarely used.

Although extensive research has gone into the mechanistic approach to HMAC materials testing and various methods exist (SHRP 1994-A, SHRP 1994-B, FHWA

2000), there is a lack of consistency and agreement in the industry as to which tests, if any, best predict pavement performance. Despite the attempts by SHRP, a standardized approach to performance prediction testing that is universally accepted has yet to be achieved (Carpenter and Vavrik 2001, Witczak *et al.* 2002, Brown *et al.* 2004).

2.7.2 Repeated Load Rapid Triaxial Testing

Repeated load triaxial testing is one form of mechanistic performance-related testing that is showing successful results in characterizing hot mix asphalt concrete mixes (Berthelot 1999, Carpenter and Vavrik 2001, Crockford *et al.* 2002, Shenoy and Romero 2002). The triaxial approach to testing materials was originally developed in 1930 for soils testing (Holtz and Kovacs 1981) and has been adapted in various forms to other materials testing.

In typical triaxial testing of bituminous materials, the sample is subjected to a dynamic axial load, usually applied in a sinusoidal or haversine wave form, with continuous radial confinement, as illustrated in Figure 2.18. Although some unconfined test methods exist, applying confining pressure better represents material field state conditions, since the asphalt concrete in vehicle wheel paths is confined in the field by the surrounding pavement structure. Also, applying confining pressure to the sample during characterization allows better representation in the field states without prematurely failing the sample (Brown *et al.* 2004). Studies to determine whether confined or unconfined tests provide better performance prediction in terms of permanent deformation have presented conflicting results (Carpenter and Vavrik 2001, Shenoy and Romero 2002, Pellinen and Witczak 2002, Sotil *et al.* 2004). However, it should be noted that unconfined tests do not provide the necessary material constitutive relations under realistic field state conditions for mechanistic road modelling; they are only an index of strength.

Sophisticated triaxial test apparatus available now is software-operated, full feedback controlled, capable of applying loads at various frequencies, with multiple combinations of axial and radial stress states, across a range of test temperatures. The equipment measures the radial and axial strains resulting from the loading combinations, and allows the quantification of elastic and visco-elastic material properties, such as the Complex Modulus (E*), Dynamic Modulus (E_d), Poisson's Ratio (ν), and the Phase Angle (δ). Studies are currently under way to evaluate repeated load triaxial test methods for inclusion in the Superpave™ mix design to predict rutting performance of HMA mixes (NCHRP 2004, NCHRP 2005).

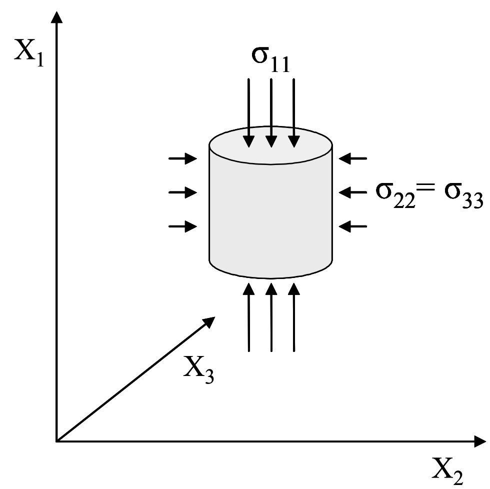

Figure 2.18 Application of Stresses in Confined Repeated Load Triaxial Testing

2.7.2.1 Complex and Dynamic Modulus

Hot mix asphalt concrete is a multi-phase particulate composite material. Due to the rheological properties of the asphalt cement binder, HMAC mixtures behave as visco-elastic solids under typical ranges of Saskatchewan field state conditions. For visco-elastic materials, the stress-strain relationship under a continuous sinusoidal loading can be defined by a complex number, E*, that is comprised of a real and an imaginary component. The real component is considered the recoverable (elastic) portion of the deformation, and the imaginary component is the non-recoverable (viscous) portion.

The Complex Modulus is a ratio of the amplitude of the time-dependent sinusoidal stress applied to the material and the amplitude of the time-dependent sinusoidal strain that results from the stress application (Pellinen and Witczak 2002). This relationship can be expressed as follows:

$$E^* = \frac{\sigma}{\varepsilon} = \frac{\sigma_{11p} e^{i\omega t}}{\varepsilon_{11p} e^{i(\omega - \delta)}} \qquad \textbf{(2.6)}$$

where:

E* = Complex Modulus (Pa)

σ = Applied stress (Pa)

ε = Strain response to applied stress (μm/μm)

σ_{11p} = Peak stress applied in the X_1 coordinate direction (Pa)

e = Exponent e

i = Imaginary component

ω = Angular load frequency (radians per second)

t = Load duration (seconds)

ε_{11p} = Peak strain response in X_1 coordinate direction (μm/μm)

δ = Phase angle (radians)

A higher stiffness modulus indicates that a given applied stress results in lower strain in the mixture (Roberts *et al.* 1996). Implemented for ease of interpretation, the dynamic modulus for linear visco-elastic materials, E_d, is a measure of the absolute value of peak stress to peak strain during material response. The primary purpose for determining the dynamic modulus is to quantify the stress-strain relationships in a pavement structure under an applied load. For an elastic material, the applied stress results in instantaneous strain, and the phase angle is zero, therefore, after manipulating equation 2.5, the dynamic modulus can be expressed as the absolute value of the complex modulus, E* (Berthelot 1999), as is illustrated in Equation 2.7.

$$E_d = \left| E^* \right| = \frac{\sigma_{11p}}{\varepsilon_{11p}} \qquad \textbf{(2.7)}$$

2.7.2.2 Phase Angle

The phase angle in a repeated load triaxial test is the shift in time between the applied stress and the resultant strain, and can be used to indicate the visco-elastic properties of the material tested, as shown in Figure 2.19 (Pellinen and Witczak 2002).

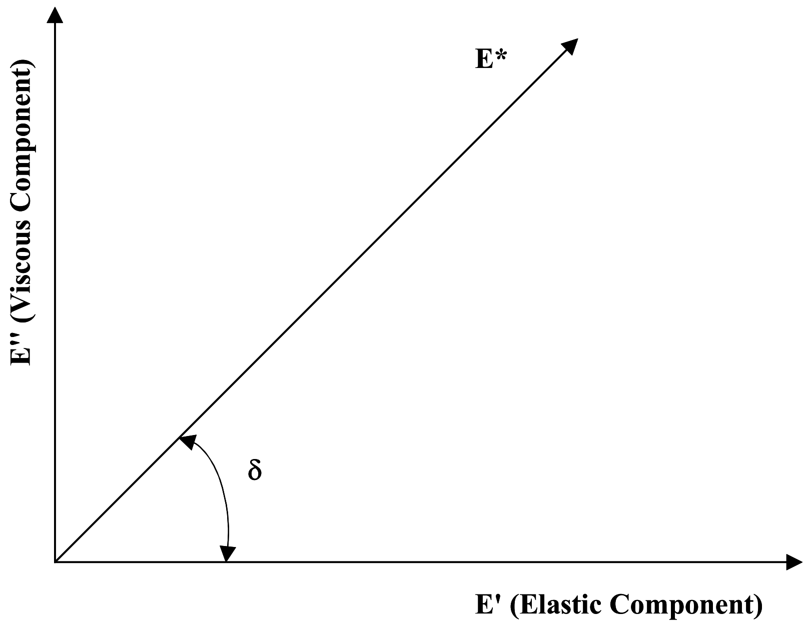

Figure 2.19 Phase Angle and Complex Modulus E* in Polar Coordinates

In a purely elastic response, the phase angle will be zero, whereas a purely viscous response will be indicated by a phase angle of 90 degrees. Phase angle can be expressed as (Yoder and Witczak 1975):

$$\delta = \frac{t_i}{t_p}(360°) \tag{2.8}$$

where:

δ = Phase Angle (degrees)

t_i = time lag between a cycle of sinusoidal stress and a cycle of strain (sec)

t_p = time for a stress cycle (sec)

2.7.2.3 Poisson's Ratio

Poisson's ratio is the relationship of the lateral strain to the axial strain, resulting from an applied load in the axial direction. When continuous radial confinement is applied to a sample in triaxial testing, radial and axial strains are monitored directly, and Poisson's ratio can be expressed as (Berthelot 1999):

$$\nu_{11}(t) = \frac{\varepsilon_{22}(t)}{\varepsilon_{11}(t)} = \frac{\varepsilon_{33}(t)}{\varepsilon_{11}(t)} \tag{2.9}$$

where:

ν = Poisson's Ratio in X_1 coordinate direction

ε_{11} = Strain in X_1 coordinate direction (axial)

ε_{22} = Strain in X_2 coordinate direction (radial)

ε_{33} = Strain in X_3 coordinate direction (radial)

Because particulate composite materials are capable of generating significant ranges in Poisson's ratio, Poisson's ratio can be a critical measure of mechanistic behaviour of road materials and can significantly influence the behaviour of road structures, depending on the material location in the road structure.

2.8 Chapter Summary

This chapter presented a discussion on the challenges faced by Saskatchewan Highways and Transportation in the area of asphalt pavements. A summary of the hot mix asphalt concrete design and evaluation concepts applicable to this research were also presented.

With increasing traffic loadings and the limited funding for the maintenance and rehabilitation of Saskatchewan highways, there is a need to deliver value-engineered asphalt concrete pavements. Saskatchewan highways have demonstrated premature pavement failures due to plastic flow rutting, and the amount of rutting on the provincial asphalt concrete road network is increasing. Other road agencies in North America have also identified plastic flow as the main cause of rutting. There is a need to design mixes that are capable of withstanding the increased traffic loadings and are not susceptible to plastic flow rutting.

In dense-graded mixes, such as the ones used by Saskatchewan Highways and Transportation, aggregate gradation, shape, angularity, and texture are key in providing a stable and structural aggregate skeleton. In light of Saskatchewan's glacial history, asphalt mix aggregates are manufactured from glacial deposits, the quality and availability of which are continually declining. Maximizing aggregate usage and at the same time providing high-performance aggregate for hot mix asphalt production is essential for SDHT.

The current Marshall mix design method employed by the SDHT utilizes a phenomenological-empirical approach to characterizing asphalt mixes. For this reason, the Marshall method does not adequately quantify the fundamental mechanistic material properties needed to properly characterize asphalt mix behaviour, and the Marshall stability and flow results are not directly related to field performance.

The Hveem mix design method, and the Superpave™ Level I mix design method are also used in North America. Although they have advantages over the Marshall method of mix design, the Hveem and the Superpave™ Level I methods do not directly measure mechanistic material properties. The mechanistic-based tests used in Superpave Level II and Level III mix design are complex, time consuming, expensive, and the debate over their ability to predict performance continues.

This research concluded that repeated load triaxial testing is showing successful results in characterizing asphalt mixes, and provides fundamental material constitutive relations necessary for mechanistic road modeling.

CHAPTER 3 CONVENTIONAL PHYSICAL AND EMPIRICAL MIX ANALYSIS OF RESEARCH MIXES

This chapter presents a summary of the physical properties of the research mixes used for this study, as well as empirical mix design properties of the research mixes which are measured as part of conventional Marshall asphalt mix analysis used by Saskatchewan Highways and Transportation. Included are physical aggregate properties of the aggregate used, the volumetric properties of the compacted asphalt concrete samples for each research mix as a function of the method of compaction, and Marshall stability and flow results.

Where applicable, statistical analysis of the test results was performed to quantify significant differences across the various mix types considered in this research. Analysis of Variance (ANOVA) is used to identify the main interaction effects of the independent variable(s) on the dependent variable(s). If significant interaction is found through ANOVA, Tukey's Homogeneous Groups comparison was selected to perform more detailed analysis, through pairwise comparison across the multiple dependent variables. This approach compares the mean of each population against the mean of each of the other populations, creating separate groups for results that are statistically different, based on a level of significance, α, of 0.05.

The final section presents the estimated level of confidence based on the ten repeat samples determined for the volumetric and Marshall properties measured, and possible experimental errors are discussed.

3.1 Physical Properties of Research Mixes

In order to evaluate the benefits of manufactured fines content in SDHT asphalt concrete mixes, asphalt concrete mix design adaptable to the manipulation of manufactured fines content without compromising volumetric properties was required. In addition, it was desirable to compare the SDHT mixes with different levels of manufactured fines with a conventional SDHT mix thought to have good structural performance (SDHT Type 70 mix). Another selection limitation was availability of the necessary amount of aggregate needed to create the samples for this research.

The HMAC paving project on Highway 11, south of Craik, was a suitable candidate for material sampling because the project design required a Type 70 SDHT asphalt concrete mix gradation on the bottom lift, and a Type 72 gradation on the top lift. Another reason for selecting this project is that both the mixes were manufactured from the same gravel source. SDHT Type 70 asphalt mix gradation is a structural mix, with an 18 mm aggregate top size and is thought to be the most rut-resistant of the three SDHT mix gradations. The Type 72 gradation is used for the top lift only, to provide a smooth, durable surface, due to its small top size (12.5 mm). Therefore, the Type 72 mix with varying amounts of manufactured fines could be compared to the Type 70 mix. In addition, the Highway 11 rehabilitation project was ongoing at the time of the design of this research; therefore, asphalt concrete material samples were readily available.

According to discussions with SDHT laboratory staff involved in mix designs, typical SDHT mixes at various locations in the province range in manufactured fines content from as low as 20 percent of total fines (passing the 5 mm sieve), to approximately 40 percent of total fines. SDHT was interested in determining the behaviour of Type 72 asphalt concrete mix with the typical amounts of manufactured fines, as well as at an increased level. Therefore, three Type 72 HMAC mix designs were created for this research, based on a mix design used for the Highway 11 project. The amount of manufactured coarse aggregate (retained on the 5 mm sieve) was maintained constant across the Type 72 mixes considered, and the manufactured fines were substituted for natural fines in order to vary the manufactured fines content only. The resulting aggregate design blends for the Type 72 mix incorporated 20, 40, and 60 percent of manufactured fine aggregate, respectively, as determined by weight on the portion of total fine aggregate (passing the 5 mm sieve). The structural Type 70 mix manufactured for the Highway 11 paving project had 38 percent fine aggregate, and was included in the study without modifications. It should be noted that typical SDHT asphalt mix aggregate gradations contain approximately 40 percent manufactured fines. For the purposes of reporting, the research mixes were named according to their gradation type and respective manufactured fines contents, as shown in Table 3.1. Appendix A contains the mix design summary for each of the mixes that were created for the purposes of this research.

The stockpile proportions used to create the research aggregate blends are illustrated in Table 3.1 and Figure 3.1. The aggregate sampled from each stockpile was sieved into individual particle sizes in the laboratory, and aggregate was recombined by

Table 3.1 Proportions of Aggregate Stockpiles in Research Mixes

Mix Type	Mix Name	Manufactured Coarse (%)	Manufactured Fines (%)	Natural Fines (%)
Type 72 (20%MF)	T72(20%MF)	31.0	14.0	55.0
Type 72 (40%MF)	T72(40%MF)	31.0	29.0	40.0
Type 72 (60%MF)	T72(60%MF)	31.0	42.5	26.5
Type 70 (38%MF)	T70(38%MF)	34.0	25.0	41.0

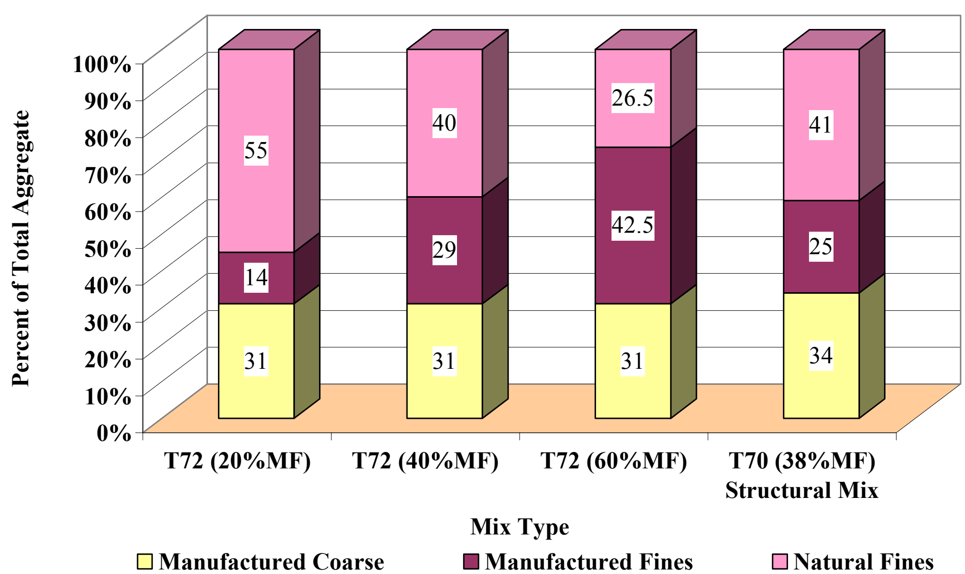

Figure 3.1 Proportions of Aggregate Stockpiles in Research Mixes

mass of each particle size from each stockpile to target the mix design gradation in order to create each repeat sample, with a tolerance of ± 0.1 grams on each sieve size. Ten repeat samples for each Marshall compaction and analysis and for the gyratory compaction and triaxial testing were created for each of the research mixes.

Canadian General Standards Board (CGSB) 150/200A asphalt cement grade was chosen to create the research mixes, because this grade of asphalt was used for the rehabilitation project on Highway 11, and was selected based on SDHT surfacing design standards (SDHT 2001-B). The 150/200A asphalt grade is the grade most commonly used for SDHT HMAC mixes. Based on previous testing of the 150/200A asphalt from the particular asphalt manufacturer who supplied the asphalt cement for the Highway 11 project, the PG grade of the asphalt cement used in this research is expected to be PG 52-28, however, this was not validated as part of this research. Due to stripping potential of the aggregates as determined by SDHT during the mix design stages for the rehabilitation project, liquid anti-stripping additive was added to create all of the research mixes.

3.1.1 Gradations of Research Mix Aggregates

The aggregate gradations of each material stockpile were separated into individual particle sizes, and samples were created in the laboratory based on gradation stockpile averages obtained from the crushing process, and according to the stockpile proportions as described in the previous section. For the gyratory compactor samples, the aggregate samples were combined for a total of 6500 ± 1 g of hot mix asphalt aggregate sample mass, whereas a 1200 ± 1 g sample size was used for the Marshall compaction samples.

The aggregate gradations of the four research mixes are shown in Table 3.2 and Figure 3.2. The Type 70 gradation closely resembles that of the Type 72 mixes on the fine side (up to the 5 mm sieve), and contains slightly coarser aggregates (greater than 5 mm). Since the SDHT gradation bands for the specified aggregate skeletons are very similar, as is illustrated in Figure 3.3 (SDHT 2003-B), the Contractor chose to create aggregate skeletons for the Type 72 and Type 70 mixes while utilizing the same

Table 3.2   Aggregate Gradations of Research Mixes

Sieve Size (mm)	Percent Passing by Weight			
	T72(20%MF)	T72(40%MF)	T72(60%MF)	T70(38%MF)
18.0	100	100	100	100
16.0	100	100	100	98.0
12.5	98.8	98.8	98.8	93.0
9.0	84.4	84.4	84.4	75.7
5.0	65.3	64.2	63.2	62.2
2.0	47.4	44.4	41.7	43.1
0.90	31.7	29.8	28.1	28.9
0.40	17.3	17.1	16.9	16.5
0.16	8.6	8.5	8.9	8.1
0.071	3.5	4.2	4.9	4.0

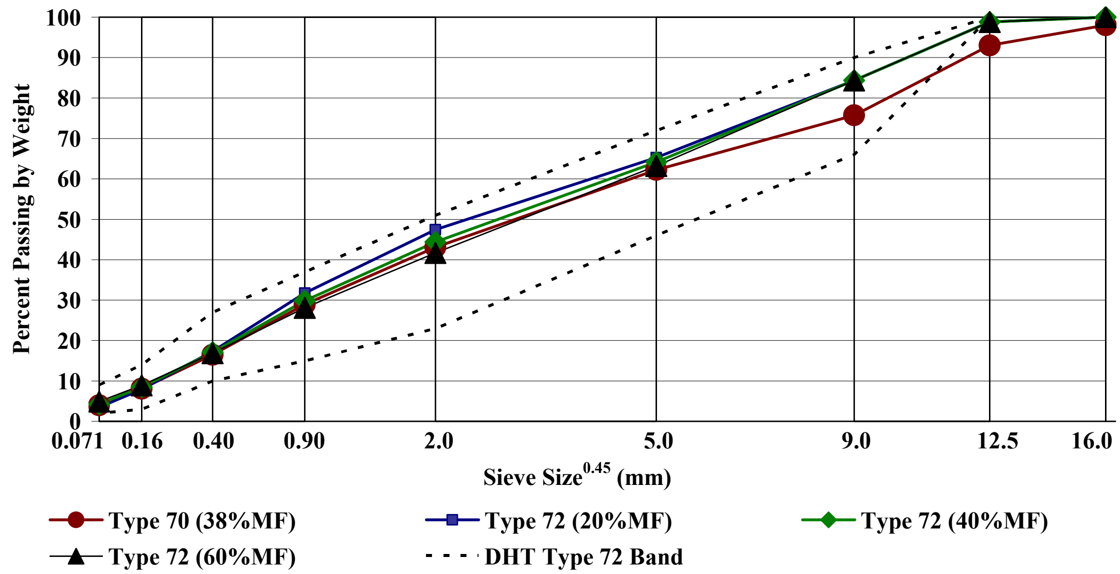

Figure 3.2 Aggregate Gradations of Research Mixes

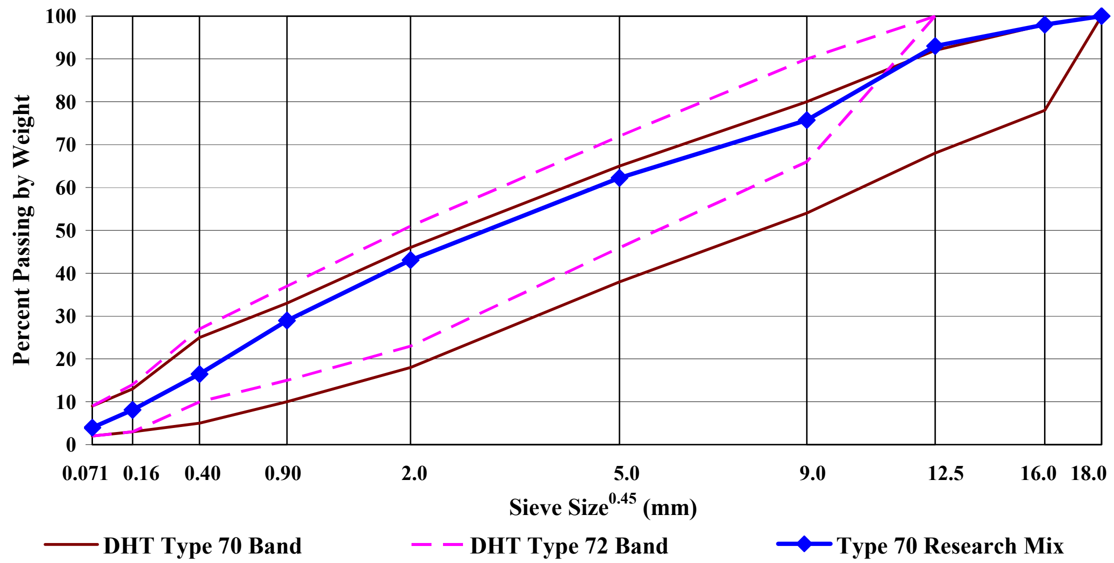

Figure 3.3 SDHT Type 70 Research Mix Aggregate Gradation

manufactured fines aggregate and natural fines aggregate stockpiles, with the only varying components being the slightly different top size of the manufactured coarse aggregate, as required for the two mix types (18 mm for Type 70, and 16 mm for Type 72). This resulted in aggregate gradations that vary mainly on the coarse end of the gradation. The similarities in particle size in the fine portion of the aggregate between the Type 72 mixes and the Type 70 mix are beneficial for this research. The research should illustrate more clearly the effects of the differences in the coarser portion of the aggregate between these two mix types.

3.1.2 Physical Properties of Research Mix Aggregates

Some of the physical properties that are generally accepted as being related to hot mix asphalt concrete design, construction, and performance behaviour are fracture in the coarse and fine aggregate, proportion of clay particles in the fine aggregate, flat and elongated particles, and lightweight materials. Prior to manufacturing the hot mix asphalt concrete samples for the purposes of this research, these physical properties were tested during the laboratory mix design stage. Table 3.3 lists the physical parameters measured, the number of repeat tests performed, and their results for each of the research aggregate blends.

Table 3.3 Physical Properties of Research Aggregates

Physical Property	No. of Tests	Mix Type			
		T72(20%MF)	T72(40%MF)	T72(60%MF)	T70(38%MF)
Coarse Fracture (%)	2	95.2	96.9	97.8	90.7
Fine Aggregate Angularity (%)	2	41.9	42.9	45.1	42.4
Sand Equivalent (%)	3	73	74	73	69
Flat & Elongated Pieces (%)	1	4.0	4.4	5.2	2.1
Lightweight Pieces (%)	2	0.2	0.3	0.2	0.2

Appendix B contains more detailed information on these test results. The number of repeat tests performed was based on standard SDHT laboratory practice, which in turn relates to achieving acceptable results, while maximizing time and monetary investment into laboratory testing. Although each of the mixes met the SDHT specifications for the parameters measured (previously discussed in Section 2.3), the number of repeat tests could have been increased for the purpose of this research, in order to provide more certainty in the results.

As can be seen in Table 3.3, percent of coarse fracture of the aggregate differs for each of the mix types, specifically, for the Type 72 mixes, it increases with the increased amount of manufactured fines, because the increase in manufactured fines content contributed to an increase in fractured aggregate retained on the 5 mm sieve. The Type 70 mix has significantly lower percent of coarse fracture when compared to the Type 72 mixes. The repeatability of this test is yet to be quantified, however, it is likely that the differences observed within a mix type are acceptable, since as previously discussed in Section 2.3.2.1, only marginal changes in mechanical stability and/or mechanical behaviour should be expected from varying the amount of fractured coarse aggregate when the coarse fracture levels are higher than 75 percent (Wedding and Gaynor 1961, Carlberg 2003). It is possible that the increased coarse fracture in the Type 72 mixes could result in an improvement of mechanical properties over the Type 70 mix; however, these are also expected to be marginal for the reasons discussed above.

Fine aggregate angularity directly reflects the changes in the manufactured fines content, which is the control measure being investigated for its influences on the research mixes. The differences in FAA across the research mixes are intentional. The sand equivalent values also differ slightly across the mixes; however, SDHT laboratory standards allow for ± four percent points of tolerance in the accuracy of this measure, and the results are within this SDHT accepted tolerance. The lower sand equivalent value for the Type 70 mix could result in a deterioration of mechanical properties.

There are currently no standards on the accuracy of the measurement of flat and elongated and the lightweight pieces. It is possible that the lower amount of flat and elongated pieces in the Type 70 mix could result in an improvement of mechanical properties.

3.1.3 Volumetric Properties of Research Mixes after Marshall Compaction

In preparation for the mechanical testing, ten repeat samples of 1200 ± 1 g were prepared for each research mix, using the Marshall compaction method. The sample mass of 1200 g is the standard used for the Marshall mix design method (ASTM D 1559, STP 204-10). The void properties for Marshall samples were determined according to SDHT Specifications for Density and Void Characteristics (ASTM D 2726, STP 204-21). The Marshall samples were compacted using 75 blows, as specified by SDHT mix design procedures (STP 204-10). The volumetric properties of the four different research mixes meet the SDHT design criteria of VTM, VMA, and VFA (previously discussed in Sections 2.4.1 to 2.4.3).

Table 3.4 and Figures 3.4 to 3.6 illustrate a summary of the mean void properties of the samples compacted using the 75 blow Marshall compaction protocol for each of the research mixes. The main bars in the figures show the mean of ten repeat samples, and the error bars represent ± two standard deviations (SD). Detailed results of volumetric properties after Marshall compaction can be found in Appendix C.

As seen in Table 3.4, the coefficients of variation (CV) for VTM are slightly higher than those for the other two parameters, ranging from 3.3 to 6.9 percent. An examination of the individual data records did not reveal any anomalies that explain the increased CVs across the research mixes. The increase appears to be caused by the smaller magnitude of VTM when compared to VMA and VFA.

There is a large amount of variability within the VFA results for Mix Type 72 with 60 percent manufactured fines, as shown by the error bars in Figure 3.6. By examining the detailed VFA results (Appendix C, Table C.3), it appears that there are two samples which are causing the high variability.

Although the variability in the Marshall void properties within each mix may play a role in the Marshall stability and flow results, it should also be noted that based on discussions with SDHT laboratory staff, the accepted level of accuracy for volumetric measurements of VMA, VTM and VFA is considered to be ± 0.2 percent for each respective parameter, therefore, keeping in mind this practical laboratory tolerance, the variability can be considered acceptable.

Analysis of variance shows that there are differences in the void properties measured between the four mix types, as indicated by the F-Test being larger than 1, and the probability factor, p, being smaller than 0.05 (Table 3.5). These differences are further explored in Table 3.6, Table 3.7 and Table 3.8, through the results of Tukey's pairwise comparison.

The Type 70 mix average VMA of 14.3 percent is statistically the same as that for the Type 72 mix with 60 percent manufactured fines (14.4 percent), and lower than the other mixes. The Type 72 mix with 20 percent manufactured fines has the highest VMA with a mean of 14.9 percent.

Table 3.4 Void Properties of Compacted Marshall Samples at 75 Blows

	Mix Type							
	T72(20%MF)		**T72(40%MF)**		**T72(60%MF)**		**T70(38%MF)**	
	Mean	**CV (%)**	**Mean**	**CV (%)**	**Mean**	**CV (%)**	**Mean**	**CV (%)**
VMA (%)	14.9	1	14.6	1	14.4	2	14.3	1
VTM (%)	4.2	5	4.1	4	4.0	7	3.9	3
VFA (%)	71.6	2	72.2	1	72.2	3	72.6	1

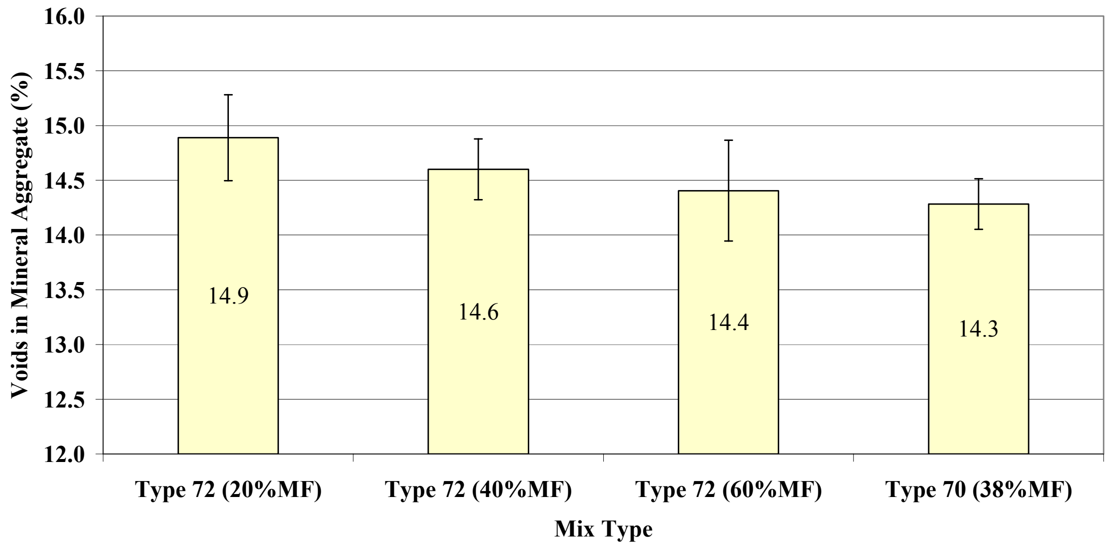

Figure 3.4 Mean Voids in Mineral Aggregate after 75 blow Marshall Compaction across Research Mixes (± 2 SD)

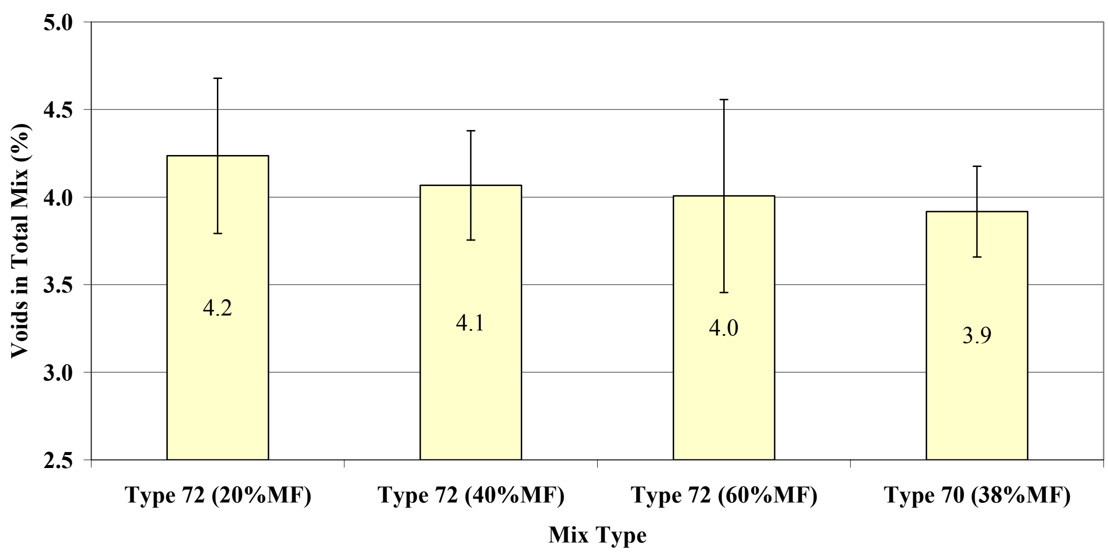

Figure 3.5 Mean Voids in Total Mix after 75 blow Marshall Compaction across Research Mixes (± 2 SD)

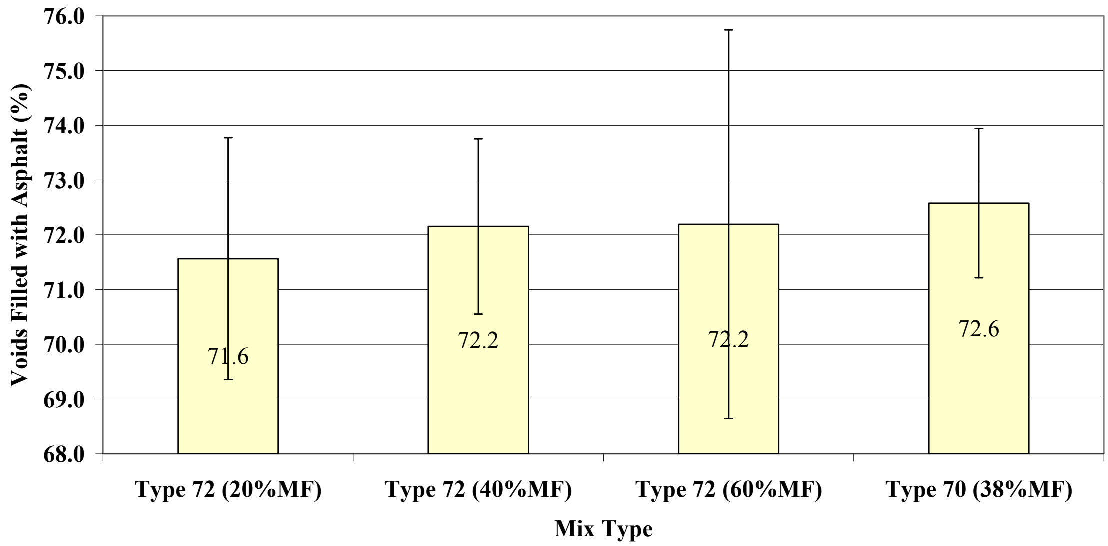

Figure 3.6 Mean Voids Filled with Asphalt after 75 blow Marshall Compaction across Research Mixes (± 2 SD)

The Type 72 mix with 20 percent manufactured fines has the highest VTM with a mean of 4.2 percent, while the lowest VTM is that for Type 70 mix at a mean of 3.9 percent. However, all the mixes have very similar VTM, as is shown by a lot of interaction in the Tukey's homogeneous groups. There are no significant differences in the VFA between the research mixes, which ranges from 71.6 percent for mix Type 72 with 20 percent manufactured fines, to 72.6 percent for mix Type 70.

Table 3.5 Analysis of Variance for Marshall Void Properties across Research Mixes

Parameter	Test	Value	F-Test Statistic	Effect	Error	P-value
Mix Type	*Wilks*	*0.2399*	*7*	*9*	*82.90*	*0.00*

Table 3.6 Tukey's Homogeneous Groups for Marshall Voids in Mineral Aggregate across Research Mixes

Mix Type	Mean VMA (%)	Tukey's Homogeneous Groups		
		A	B	C
T70(38%MF)	14.3	****		
T72(60%MF)	14.4	****	****	
T72(40%MF)	14.6		****	
T72(20%MF)	14.9			****

Table 3.7 Tukey's Homogeneous Groups for Marshall Voids in Total Mix across Research Mixes

Mix Type	Mean VTM (%)	Tukey's Homogeneous Groups	
		A	B
T70(38%MF)	3.9	****	
T72(60%MF)	4.0	****	****
T72(40%MF)	4.1	****	****
T72(20%MF)	4.2		****

Table 3.8 Tukey's Homogeneous Groups for Marshall Voids Filled with Asphalt across Research Mixes

Mix Type	Mean VFA (%)	Tukey's Homogeneous Groups
		A
T72(20%MF)	71.6	****
T72(40%MF)	72.2	****
T72(60%MF)	72.2	****
T70(38%MF)	72.6	****

3.1.4 Volumetric Properties of Research Mixes after Gyratory Compaction

Although the mass of aggregate combined according to the stockpile average for preparation of the asphalt samples was 6500 ± 1 g, the sample mass used for gyratory compaction was adjusted based on a desired final sample height of 150 ± 5 mm, which is necessary for the mechanistic testing equipment used as part of this study. Ten repeat samples were compacted for each mix using the gyratory compaction method, with a sample mass of 6250 ± 1 g for Mix Type 72 with 40 percent manufactured fines, and with 6267 ± 1 g of HMAC per sample for the other three mixes.

One of the many benefits of the gyratory compactor is the ability to monitor and record the volumetric changes in the sample during compaction. Once the samples are compacted, their volumetric properties are also determined using the standard method of bulk specific gravity of the compacted mix (G_{mb}) by weight in water (ASTM D 2726). A correction factor consisting of the ratio of the gyratory bulk specific gravity and the ASTM bulk specific gravity is applied to the volumetric results of each sample. All of the gyratory volumetric properties reported in this section have been corrected in this manner. The gyratory compaction was conducted according to Superpave™ testing protocols, with N_{design} of 96 gyrations and $N_{maximum}$ of 152 gyrations (AASHTO TP-4). Detailed results of volumetric properties during and after gyratory compaction can be found in Appendix D.

Table 3.9 shows the volumetric properties for samples compacted using the gyratory compactor at N_{design}. Each of the volumetric properties is illustrated in Figures 3.7 to 3.9.

Table 3.9 Void Properties of Compacted Gyratory Samples at N_{design}

	Mix Type							
	T72(20%MF)		T72(40%MF)		T72(60%MF)		T70(38%MF)	
	Mean	CV (%)	Mean	CV (%)	Mean	CV (%)	Mean	CV (%)
VMA (%)	14.3	1	14.4	3	14.2	2	13.8	2
VTM (%)	3.3	5	3.6	12	3.4	7	3.1	10
VFA (%)	76.8	1	75.4	3	76.0	2	77.7	2

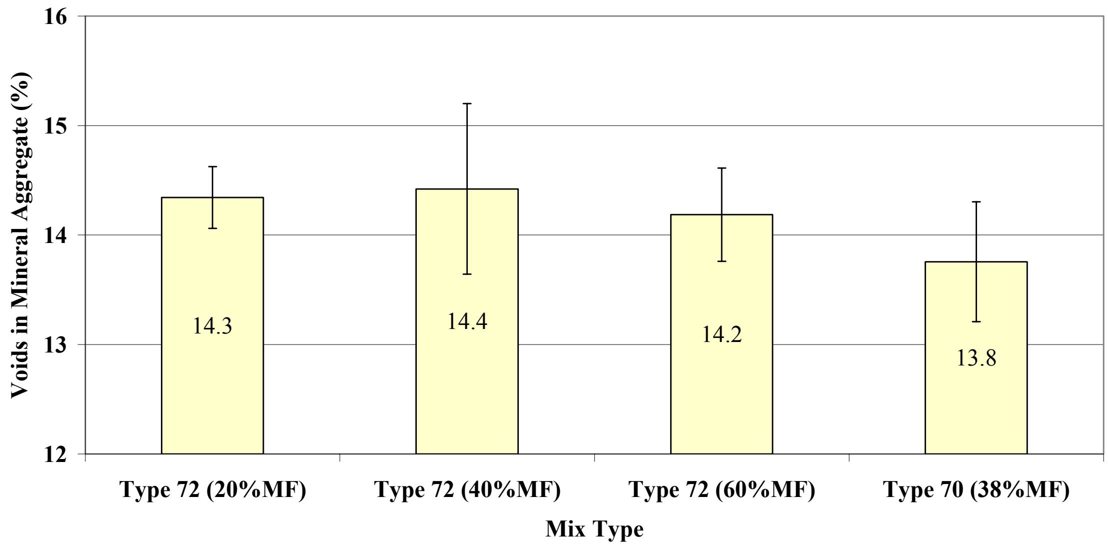

Figure 3.7 Mean Voids in Mineral Aggregate after Gyratory Compaction to N_{design} across Research Mixes (± 2 SD)

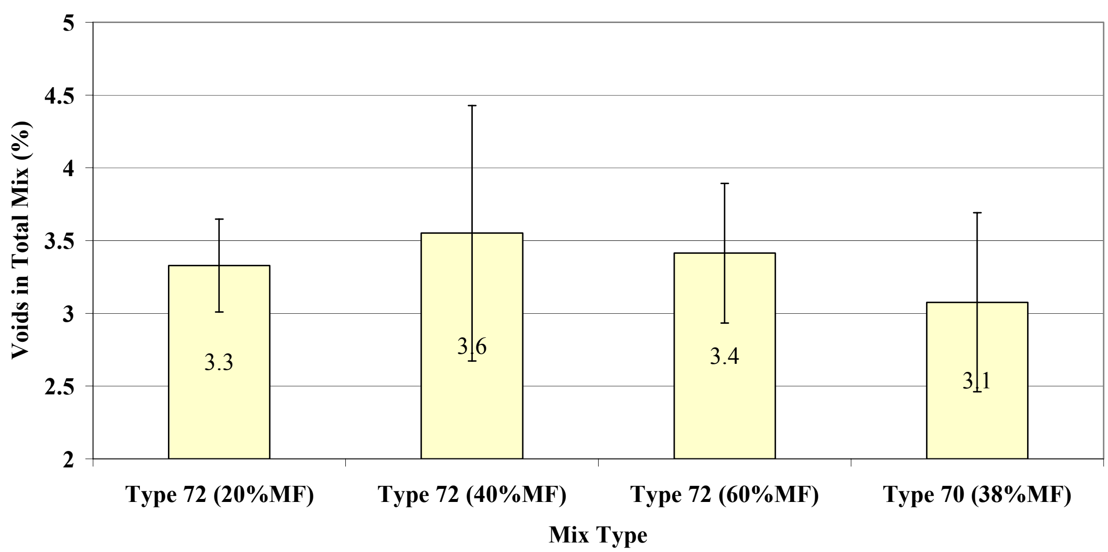

Figure 3.8 Mean Voids in Total Mix after Gyratory Compaction to N_{design} across Research Mixes (± 2 SD)

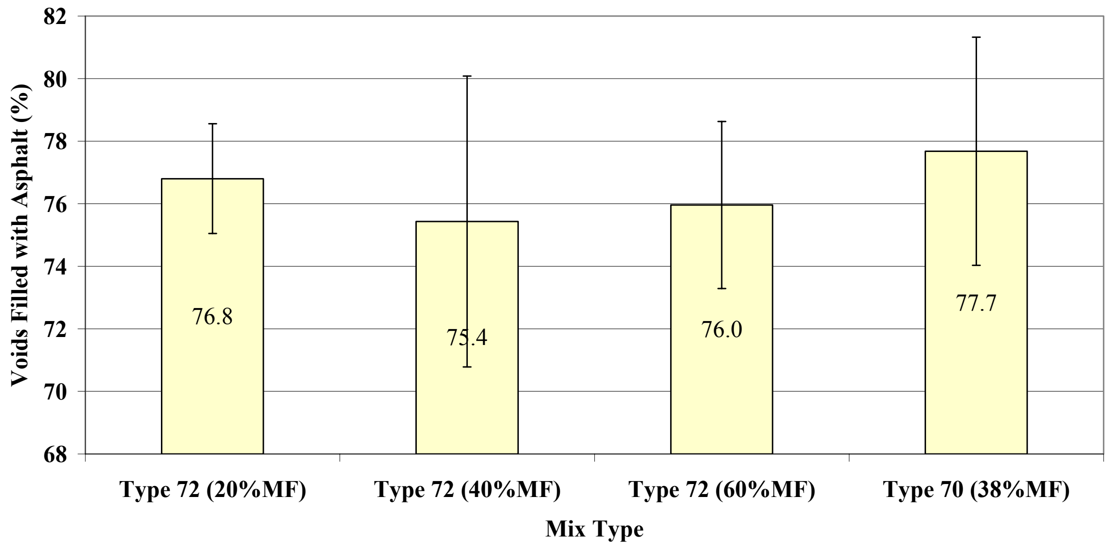

Figure 3.9 Mean Voids Filled with Asphalt after Gyratory Compaction to N_{design} across Research Mixes (± 2 SD)

The coefficients of variation for VTM are slightly higher than those for VMA and VFA, for each of the four research mixes. This is consistent with the observed CVs in the Marshall compacted samples, and appears to be related to the smaller magnitude of VTM when compared to VMA and VFA. As can be seen in the graphical illustrations, there appears to be considerable variability in the results for some of the mixes compacted in the gyratory compactor to N_{design}. The coefficients of variation are higher than expected, especially for the VTM of the Type 72 mix with 40 percent manufactured fines (CV of 12 percent) and the Type 70 mix (CV of 10 percent). Normally, the gyratory compactor is thought to provide highly repeatable results – discussion on the observed variability is presented in Section 3.3.4.

Analysis of variance across the void properties (Table 3.10) indicates that there are significant differences in the volumetric parameters between mix types. These differences are further illustrated in Tables 3.11 to 3.13 in the results of Tukey's pairwise comparison.

Table 3.10 Analysis of Variance for Gyratory Void Properties at N_{design} across Research Mixes

Parameter	Test	Value	F-Test Statistic	Effect	Error	P-value
Mix Type	*Wilks*	*0.000002*	*2.010E+03*	*9*	*82.897*	*0.00*

Table 3.11 Tukey's Homogeneous Groups for Gyratory Voids in Mineral Aggregate at N_{design} across Research Mixes

Mix Type	Mean VMA (%)	Tukey's Homogeneous Groups	
		A	B
T70(38%MF)	13.8	****	
T72(60%MF)	14.2		****
T72(20%MF)	14.3		****
T72(40%MF)	14.4		****

Table 3.12 Tukey's Homogeneous Groups for Gyratory Voids in Total Mix at N_{design} across Research Mixes

Mix Type	Mean VTM (%)	Tukey's Homogeneous Groups	
		A	B
T70(38%MF)	3.1	****	
T72(20%MF)	3.3	****	****
T72(60%MF)	3.4	****	****
T72(40%MF)	3.6		****

Table 3.13 Tukey's Homogeneous Groups for Gyratory Voids Filled with Asphalt at N_{design} across Research Mixes

Mix Type	Mean VFA (%)	Tukey's Homogeneous Groups	
		A	B
T72(40%MF)	75.4	****	
T72(60%MF)	76.0	****	****
T72(20%MF)	76.8	****	****
T70(38%MF)	77.7		****

The Type 70 mix yielded lower VMA than the three Type 72 mixes (mean of 13.8 percent), similar to the Marshall samples, while there is no significant difference in VMA between the three Type 72 mixes. In terms of VTM, the only significant difference noted is between the Type 70 mix (mean VTM of 3.1 percent) and the Type 72 mix with 40 percent manufactured fines (mean VTM of 3.6 percent). Similarly, the only significant difference in VFA results is also between the Type 70 mix (mean FVA of 77.7 percent) and the Type 72 mix with 40 percent manufactured fines (mean VFA of 75.4 percent).

3.1.5 Densification of Research Mixes during Gyratory Compaction

Superpave™ Level 1 mix design imposes restrictions on the densification rate of the asphalt mix by specifying a maximum percent of densification at initial compaction ($N_{initial}$ = 8 gyrations), and after the final compaction ($N_{maximum}$ = 152 gyrations). The specifications limit the ratio of the specific gravity of the mix with respect to the maximum theoretical specific gravity (G_{mm}) to less than 89 percent at $N_{initial}$, and to less than 98 percent at $N_{maximum}$. Table 3.14 shows the mean values of ten repeat samples for the percent of maximum theoretical specific gravity achieved at each milestone level of compaction for each of the four research mixes. This data is also illustrated in Figure 3.10.

Table 3.14 Mean Densification of Research Mixes during Gyratory Compaction expressed as Percent Maximum Theoretical Specific Gravity (%G_{mm})

	Mix Type							
	T72(20%MF)		T72(40%MF)		T72(60%MF)		T70(38%MF)	
	Mean	CV (%)	Mean	CV (%)	Mean	CV (%)	Mean	CV (%)
% G_{mm} at $N_{initial}$	92.3	0.9	89.3	0.6	88.9	0.7	90.1	0.4
% G_{mm} at N_{design}	96.7	0.2	96.4	0.5	96.6	0.2	96.9	0.3
% G_{mm} at $N_{maximum}$	97.5	0.2	97.4	0.4	97.7	0.2	97.8	0.3

Increasing the amount of manufactured fines in the Type 72 mix resulted in progressively less densification in the mix, with the mix passing Superpave™ specification of less than 89 percent of G_{mm} at $N_{initial,}$ based on the average across the ten repeat samples, when the manufactured fines content was increased to 60 percent of total fines (mean percent G_{mm} of 88.9 percent). However, even this mix had samples which failed the $N_{initial}$ specification, based on the error bars, which represent two standard deviations. Failure to meet the $N_{initial}$ criterion by the other three mixes indicates that these mixes may prove problematic during construction (i.e. a tender mix), and may be susceptible to collapsed air voids and therefore to permanent deformation.

Superpave™ mix design method aims for four percent VTM at N_{design}. The corresponding level of G_{mm} that results in four percent VTM is 96 percent. As can be seen in Figure 3.10, all of the research mixes compacted to higher percent G_{mm} at N_{design} than 96 percent, therefore resulting in average VTM slightly lower than the desirable four percent.

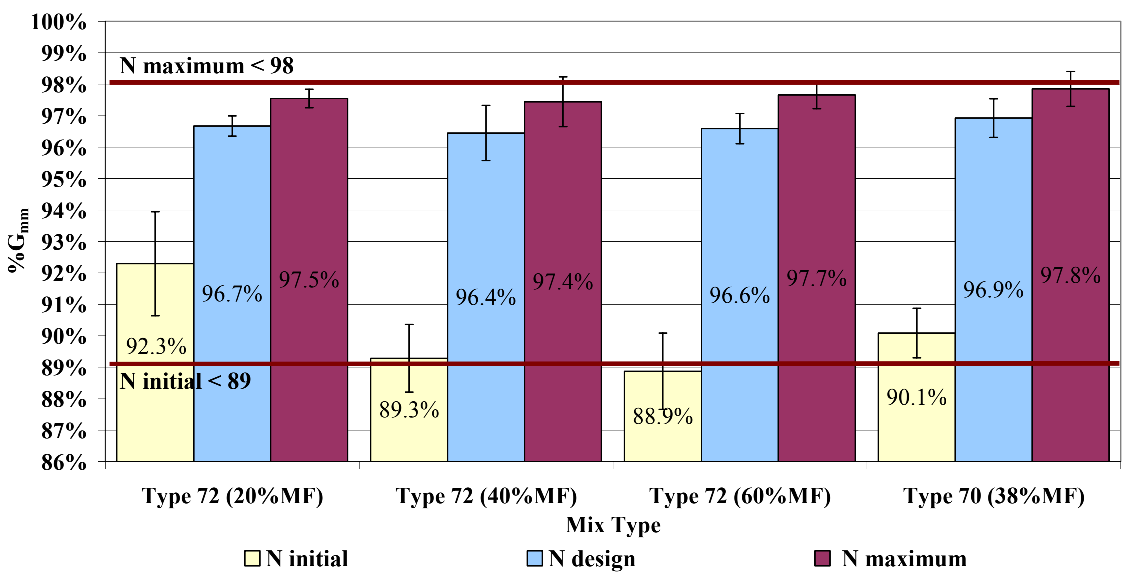

Figure 3.10 Mean Densification of Research Mixes during Gyratory Compaction expressed as Percent Maximum Theoretical Specific Gravity (%G_{mm}) (± 2 SD)

3.1.6 Comparison of Marshall and Gyratory Compaction Results

Figure 3.11 shows a comparison of the average Voids in Total Mix for each of the research mixes and the two different compaction methods, with respect to SDHT mix design criteria for VTM, and Superpave^{TM} recommended design level of air voids.

As can be seen, the average VTM for the 75-blow Marshall compacted samples met SDHT design criteria of 3 to 5 percent VTM for each of the research mixes (ranging from 3.9 to 4.2 percent), and resulted in VTM very close to the Superpave™ recommended target of 4 percent. Although the gyratory compacted samples for each of the research mixes on average met the SDHT design criteria for VTM at the N_{design} level of compaction (ranging from 3.1 to 3.6 percent across research mixes), in general the samples compacted to a lower than the acceptable design level of four percent suggested by the Superpave™ mix design process. Also, at $N_{maximum}$ the gyratory samples for each of the research mixes on average compacted below the SDHT acceptable level of 3 percent (ranging from 2.2 to 2.6 percent across research mixes). This difference in the level of compaction between the Marshall and gyratory methods was expected, since it is generally accepted that the gyratory compaction protocol results in a higher level of compaction.

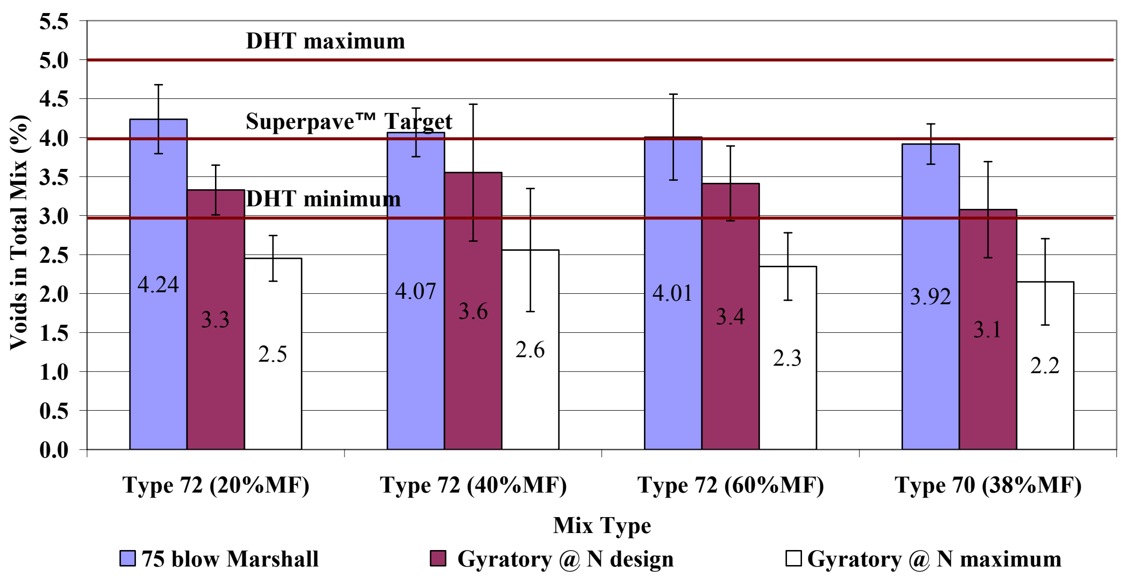

Figure 3.11 Comparison of Mean Voids in Total Mix between Marshall and Gyratory Compacted Samples (± 2 SD)

It should also be noted that normally the gyratory compaction protocol results in high repeatability. In this case, however, the Marshall samples have a lower standard deviation than the gyratory samples. This is likely due to the fact that the Marshall samples were compacted by professional and certified SDHT laboratory staff, whereas the gyratory compaction was carried out by the author, with significantly less laboratory experience. The resulting variability in the gyratory sample air voids highlights the critical importance of strict adherence to laboratory testing protocols to increase repeatability.

3.2 Marshall Characterization of Research Mixes

The first phase of determining the behaviour of the asphalt mixes in this research consisted of conducting the conventional mechanical tests included in the Marshall mix design method, namely Marshall stability and flow tests (ASTM D 1559, AASHTO T 245, STP 204-10). The Marshall mix design method is currently used by Saskatchewan Highways and Transportation in hot mix asphalt design and construction. This section presents the test results from the Marshall stability and flow tests performed on ten repeat specimens for each of the four research mixes, compacted using 75 blows of Marshall compaction. All charts illustrate the mean values of ten repeat samples tested, with the error bars representing ± two standard deviations from the mean.

3.2.1 Marshall Stability

SDHT specifies a minimum acceptable Marshall stability of 7,000 Newton for a 75 blow Marshall mix. As can be seen in Table 3.15and Figure 3.12, when considering the mean results, all four research mixes met the SDHT design criteria for Marshall stability. However, one of the repeat samples for the Type 72 mix with 20 percent manufactured fines did not. Detailed results of Marshall stability testing are presented in Appendix E.

By using the Type 72 mix with 20 percent manufactured fines as a baseline, it is apparent that there is an increase in Marshall stability with an increase in manufactured fines content. Specifically, Marshall stability increased by 22 percent at 40 percent manufactured fines, and by 36 percent when the manufactured fines content was increased to 60 percent of total fines. Marshall stability for the Type 70 mix was 22 percent higher than the baseline. The stability behaviour coincides with conventional belief that when all other factors are equal, mixes with higher fracture are more “stable” when subject to loading. Another observation worth noting is the fact that the Type 70 mix, which has a larger top size of aggregate and slightly higher content of coarse aggregate than the Type 72 aggregate skeleton, results in the same stability as the Type 72 mix with 40 percent manufactured fines.

Table 3.15 Mean Marshall Stability across Research Mixes

Mix Type	Mean Marshall Stability (Newton)	Coefficient of Variation (%)	% Difference from T72(20%MF)
T72(20%MF)	8,244	10	---
T72(40%MF)	10,084	7	22%
T72(60%MF)	11,181	5	36%
T70(38%MF)	10,069	6	22%

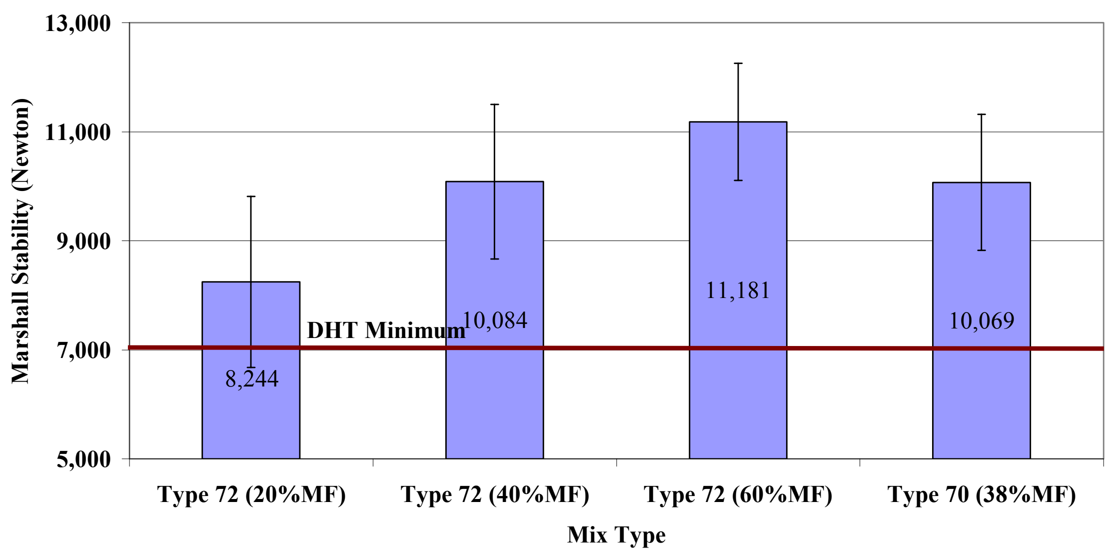

Figure 3.12 Mean Marshall Stability across Research Mixes (± 2 SD)

Statistical analysis confirms that the changes in Marshall stability are significant across the three Type 72 mixes; therefore, it was concluded that Marshall stability increases significantly as the proportion of manufactured fines increases. Type 70 mix results for Marshall stability are statistically the same as those for the Type 72 mix with 40 percent manufactured fines.

Table 3.16 Tukey's Homogeneous Groups for Marshall Stability across Research Mixes

Mix Type	Mean Marshall Stability (Newton)	Tukey's Homogeneous Groups		
		A	B	C
T72 (20%MF)	8,244	****		
T70 (38%MF)	10,069		****	
T72 (40%MF)	10,084		****	
T72 (60%MF)	11,181			****

3.2.2 Marshall Flow

The acceptable range of Marshall flow specified by SDHT is 1.5 to 3.5 mm. In Table 3.17 and Figure 3.13, it can be seen that the mean values of the ten repeat samples across all four research mixes, although close to the lower acceptable limit, met the SDHT design criteria (ranging from 1.8 to 2.3 mm across research mixes). However, there is a significant amount of variability in the test results, especially for Type 72 mix with 20 percent manufactured fines (CV of 18 percent), and for the Type 70 mix (CV of 16 percent), as is indicated by the fact that the lower error bar of two standard deviations results in the mixes failing the minimum SDHT criterion. It is also apparent that there is an increasing trend in Marshall flow with increasing amounts of manufactured fines. Specifically, there is an increase of 28 percent for the Type 72 mix with 60 percent manufactured fines. Detailed results of Marshall flow testing are shown in Appendix E.

Table 3.17 Mean Marshall Flow across Research Mixes

Mix Type	Mean Marshall Flow (mm)	Coefficient of Variation (%)	% Difference from T72(20%MF)
T72(20%MF)	1.8	18	---
T72(40%MF)	1.9	11	6%
T72(60%MF)	2.3	14	28%
T70(38%MF)	1.9	16	6%

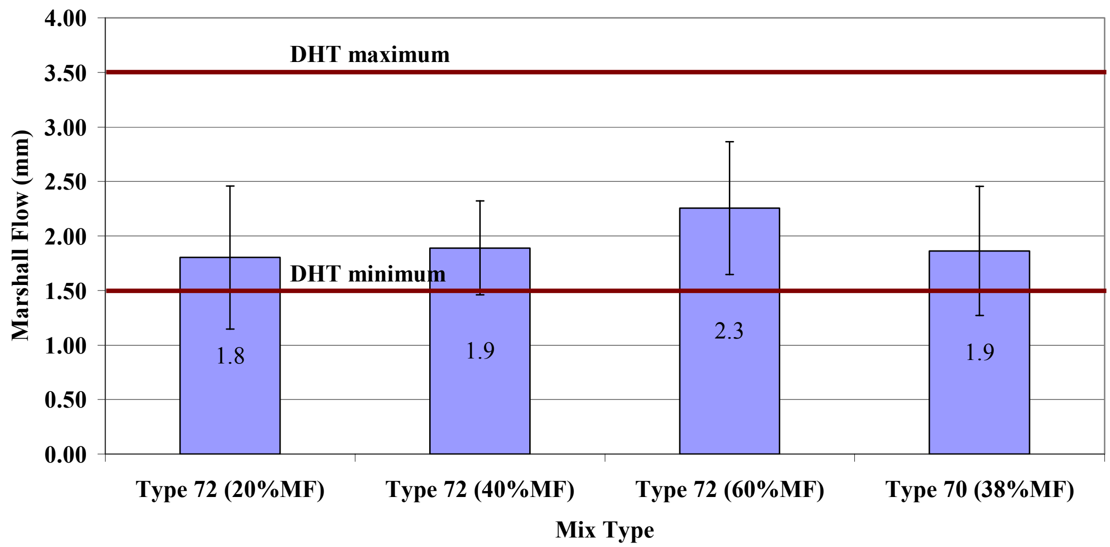

Figure 3.13 Mean Marshall Flow across Research Mixes (± 2 SD)

The results of Tukey's pairwise comparison for Marshall flow across mix type are shown in Table 3.18. The analysis shows that Marshall flow for the Type 72 mix with 60 percent manufactured fines was significantly higher than the Marshall flow for the remaining mix types. Marshall flow is not sensitive to the amount of manufactured fines below 40 percent of total fines, but there is a statistically significant difference in behaviour for the mix with 60 percent manufactured fines content.

Table 3.18 Tukey's Homogeneous Groups for Marshall Flow across Research Mixes

Mix Type	Mean Marshall Flow (mm)	Tukey's Homogeneous Groups	
		A	B
T72 (20%MF)	1.8	****	
T72 (40%MF)	1.9	****	
T70 (38%MF)	1.9	****	
T72 (60%MF)	2.3		****

3.3 Significance of Results

When determining the number of repeat samples to be used in this research, the statistical significance of the results as well as the cost and schedule associated with the proposed testing protocol had to be considered. Although manufacturing a large number of samples would achieve more statistically meaningful results, creating more than ten repeat samples was cost-prohibitive for this research. Therefore, ten repeat samples were created with the Marshall method of compaction for each of the research mixes, as well as ten repeat samples for each research mix were compacted in the gyratory compactor for the purposes of the triaxial frequency sweep analysis. It should be noted that industry standard practice is to use two repeat samples (AASHTO T 245, ASTM D 1559, STP 204-10).

3.3.1 Sample Size Analysis

In order to determine the sample size required for a desired level of significance in an experiment, a trial data set can be evaluated, and the required sample size can be estimated based on the variability within the trial data set (Sullivan 2004). When the sample size is smaller than 30, as is the case in this research study, this method can still be used, but with the assumption that the variability of the variable of interest is normally distributed, and the standard deviation of the population, σ, can be estimated by the standard deviation of the trial set, s. Assuming the above is true, the sample size required to achieve the desired margin of error at a specified level of significance can be estimated as:

$$n = \left(\frac{Z_{\alpha/2} * \sigma}{E} \right)^2 \tag{3.1}$$

where:

n = Sample size

$Z_{\alpha/2}$ = Standard normal random variable Z corresponding to $\alpha/2$

α = Level of significance

σ = Standard deviation of the population

E = Margin of error around the mean

3.3.2 Relationship of Level of Confidence to Sample Size

The level of confidence (LOC) is the probability that represents the percentage of intervals that will contain the mean if a large number of repeated samples are obtained. The level of confidence can be expressed in terms of the level of significance (Sullivan 2004):

$$LOC = (1 - \alpha) * 100\% \qquad \textbf{(3.2)}$$

To illustrate the application of these relationships, consider a desired level of confidence of 95 percent that the mean VTM for the ten repeat Marshall samples of the Type 72 mix with 60 percent manufactured fines are within 0.2 percent VTM of the population mean VTM. The mean VTM for the ten repeat samples for this mix type was 4.0 percent, as shown in Figure 3.5, with a standard deviation of 0.28 percent (Appendix C, Table C.3). Based on Equation 3.2, the level of significance, α, is 0.05, and the corresponding Z statistic is 1.96, and the number of samples required to achieve this level of confidence can be found by substituting the Z statistic and the standard deviation into Equation 3.1:

$$n = \left(\frac{1.96 * 0.28}{0.2} \right)^2 = 7$$

The margins of error used in this analysis are shown in Table 3.19. Due to lack of documented values on acceptable margins of error, these tolerances were obtained from discussions with SDHT laboratory staff, and are based on expert opinion (Bray 2006).

Table 3.19 Acceptable Margin of Error for Conventional Mix Design Properties

Property Measured	Acceptable Margin of Error
Voids in Total Mix, VTM (%)	0.2
Marshall Stability (N)	500
Marshall Flow (mm)	0.2

The sensitivity of the level of confidence to sample size based on the VTM results for Marshall samples and VTM for gyratory samples at $N_{design,}$ for each of the research mixes, is illustrated in Figure 3.14 and Figure 3.15, respectively. The relationships of LOC to sample size based on Marshall stability and flow results for each of the research mixes are shown in Figure 3.16 and Figure 3.17, respectively.

As can be seen in Figures 3.14 through 3.17, the relationship of sample size and level of confidence is exponential in nature for levels of confidence higher than 90 percent.

The gyratory VTM results demand a much larger sample size for a given desired level of confidence than the VTM for the Marshall compacted samples. This occurs due to the larger standard deviations within the ten repeat samples for the gyratory compacted samples of each of the research mixes. SDHT currently uses two repeat samples for the Marshall mix design, which based on the Marshall VTM, corresponds to a minimum level of confidence of approximately 70 percent. For the gyratory compacted samples, due to the increased variability in void properties, the minimum estimated level of confidence for three repeat samples is approximately 50 percent. Similarly, two repeat samples would achieve an estimated minimum level of confidence of 62 percent based on Marshall stability, and approximately 62 percent based on Marshall flow.

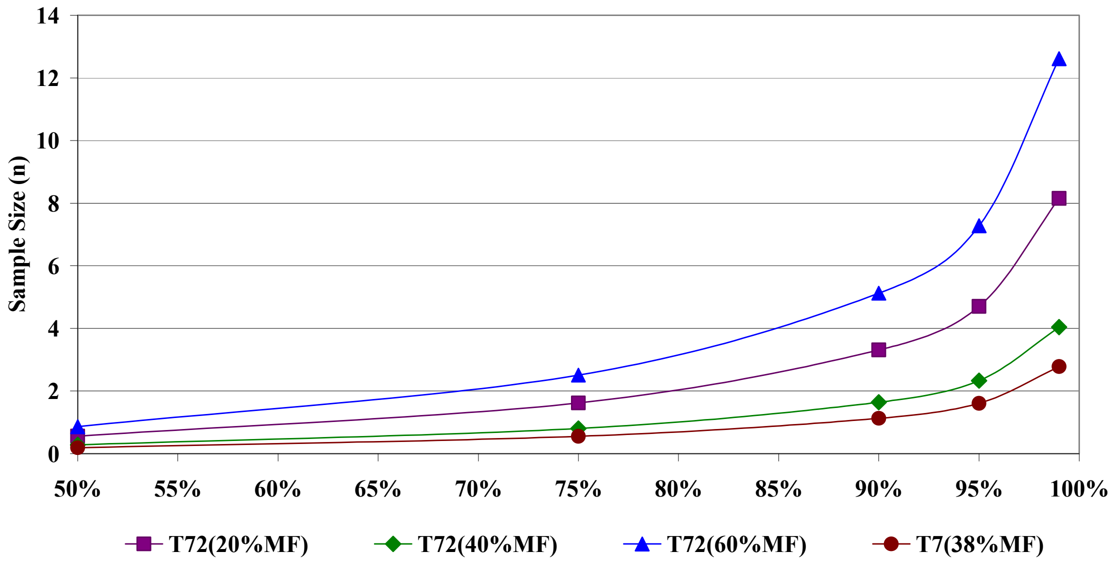

Figure 3.14 Relationship of Sample Size and Level of Confidence for Marshall Voids in Total Mix across Research Mixes at a Margin of Error of 0.2%

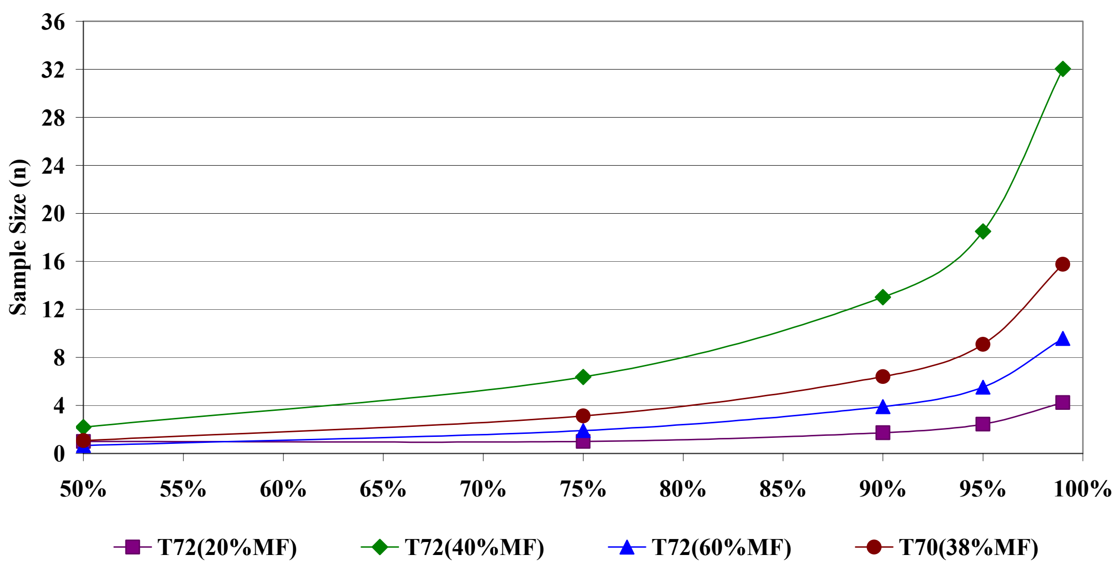

Figure 3.15 Relationship of Sample Size and Level of Confidence for Gyratory Voids in Total Mix at N_{design} across Research Mixes at a Margin of Error of 0.2%

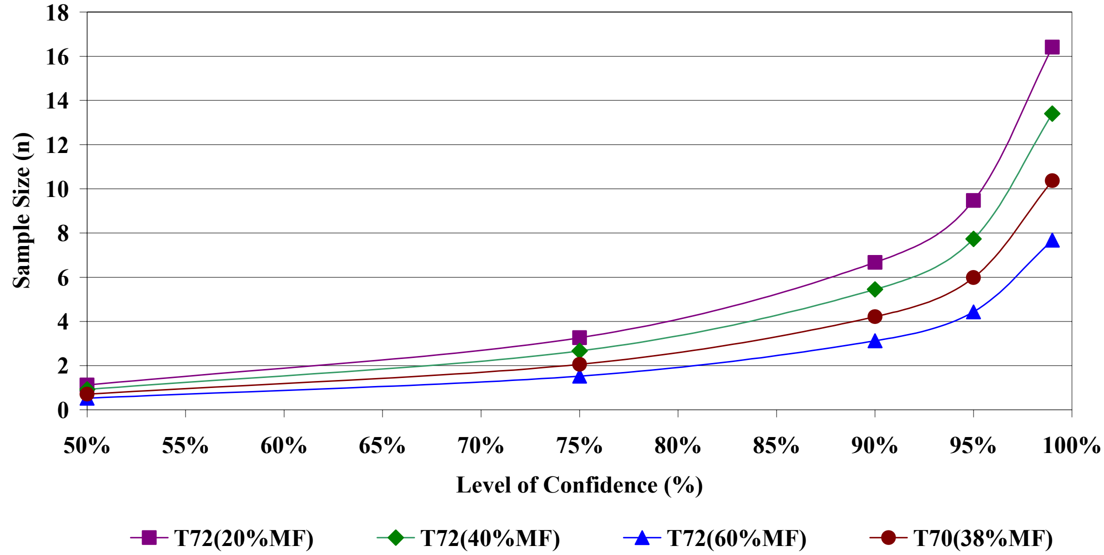

Figure 3.16 Relationship of Sample Size and Level of Confidence for Marshall Stability across Research Mixes at a Margin of Error of 500 Newton

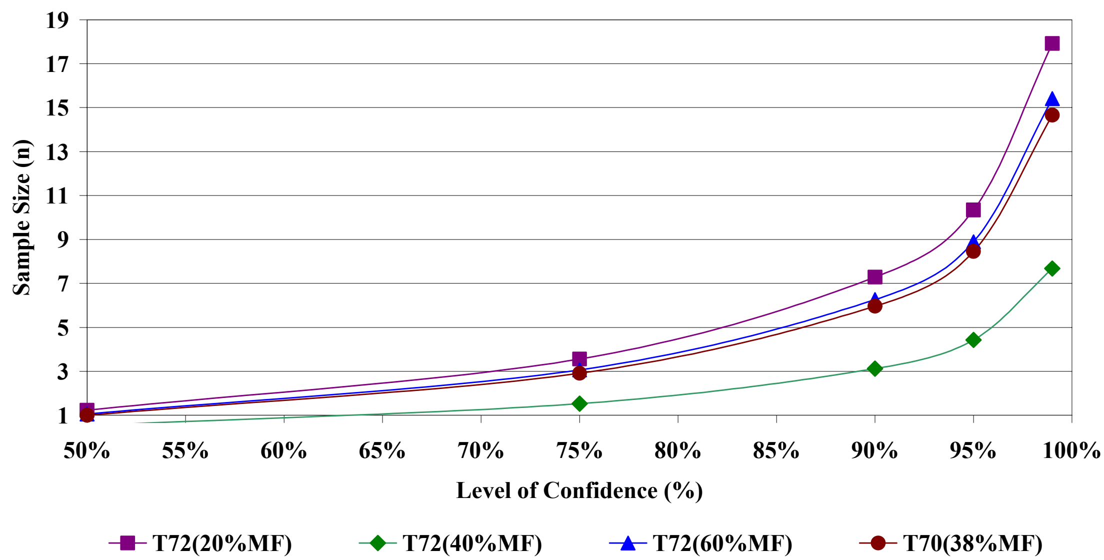

Figure 3.17 Relationship of Sample Size and Level of Confidence for Marshall Flow across Research Mixes at a Margin of Error of 0.2 mm

3.3.3 Level of Confidence Achieved

In order to estimate the level of confidence in the results achieved by using a set number of repeat samples, the sample size relationship can also be expressed as is demonstrated in Equation 3.3, which allows the calculation of the Z statistic:

$$Z_{\alpha/2} = \left(\frac{\sqrt{n} * E}{\sigma} \right) \qquad \textbf{(3.3)}$$

Once the Z statistic is calculated, the Standard Normal Distribution Table (found in Appendix F) can be used to determine the corresponding probability, $p_{\alpha/2}$,which in turn allows to estimate the achieved level of confidence, since:

$$LOC = (2 * p_{\alpha/2}) * 100\% \qquad \textbf{(3.4)}$$

To illustrate this calculation, consider the Type 72 mix with 40 percent manufactured fines. Based on Marshall compaction of ten repeat samples, the mean Voids in Total Mix (VTM) was 3.55 percent, as shown in Table 3.9, with a standard deviation of 0.44 percent (Appendix C, Table C.2). The margin of error, E, is 0.2 percent, as determined based on discussions with SDHT laboratory staff (represents approximately 5 percent of mean). Then, using Equation 3.3:

$$Z_{\alpha/2} = \left(\frac{\sqrt{10} * 0.2}{0.44} \right) = 1.44$$

The corresponding probability for the above calculated Z value of 1.44 is 0.4251 (from Standard Normal Distribution Probability Table shown in Appendix F), and therefore $\alpha/2 = 0.5 - p = 0.0749$, and the LOC can be found using Equation 3.4:

Level of Confidence = (1-(0.0749*2)*100 = 85 %

Using the approach explained in Equations 3.2 to 3.4 to quantify the significance of test results, Table 3.20 shows the estimated level of confidence based on the ten repeat samples for the VTM of Marshall samples, the VTM of the gyratory samples at

N_{design}, as well as for the Marshall stability and flow measurements for each mix. This data is illustrated in Figure 3.18.

Table 3.20 Level of Confidence Achieved for Volumetric and Marshall Properties across Research Mixes

Property Measured	Margin of Error	Estimated Level of Confidence (%)			
		T72(20%MF)	T72(40%MF)	T72(60%MF)	T70(38%MF)
$VTM_{75\ blow}$ (%)	0.2	99.6	100.0	97.9	100.0
VTM N_{design} (%)	0.2	100.0	85.0	99.2	96.1
Stability (N)	500	95.6	97.4	99.7	98.9
Flow (mm)	0.2	94.6	99.7	96.2	96.7

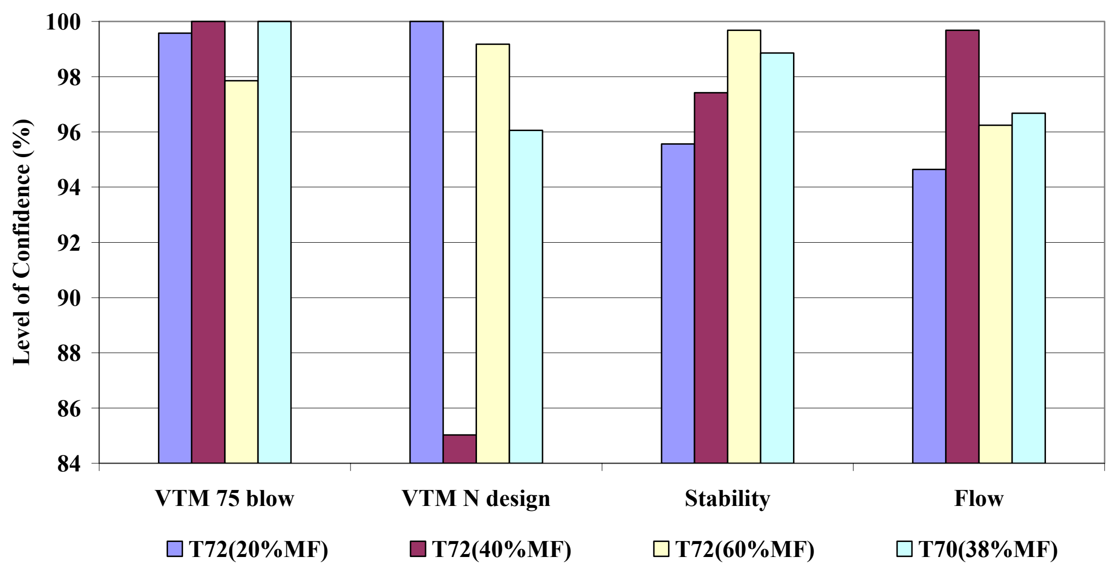

Figure 3.18 Level of Confidence Achieved for Volumetric and Marshall Properties across Research Mixes

Selecting the appropriate margin of error is critical for meaningful results of the level of confidence analysis. For example, although the Marshall flow results had a relatively high coefficient of variation within each set of repeat samples (16 to 18 percent) when compared to the volumetric test results (3 to 7 percent), because the margin of error acceptable to SDHT staff is approximately 0.2 mm, or approximately ten percent of the allowable range (SDHT specifies an acceptable range of 1.5 to 3.5 mm), the resulting level of confidence is actually very high. Also, if the margin of error for VTM were to be lowered from 0.2 percent of VTM to 0.1 percent of VTM, the resulting levels of confidence would decrease as much as 30 percent.

The parameter of greatest importance are the Voids in Total Mix, because the volumetric make-up of the asphalt mix directly impacts the mix performance in response to laboratory mechanical testing and to loading under field state conditions. For the Marshall samples, the SDHT laboratory staff was able to achieve levels of confidence above 95 percent for each parameter tested, as shown in Figure 3.18, which is considered acceptable by most researchers. In the gyratory samples, the author was able to achieve a minimum level of confidence of 85 percent based on the VTM at N_{design}, as shown in Figure 3.18. The fact that the volumetric properties of the Type 72 mix with 40 percent manufactured fines resulted in a level of confidence significantly lower than that of the other three mixes compacted in this research may suggests inconsistencies in the gyratory compaction process across repeat samples for each of the research mixes. For instance, the time to weigh and compact samples may have resulted in differential temperature at the time of gyratory compaction.

3.3.4 Experimental and Systematic Errors

The level of experience and attention to detail by the laboratory personnel can significantly affect the number of repeat samples required as well as the accuracy and precision of test results, as was illustrated in this research when the variation in volumetric properties after Marshall and gyratory compaction was compared. Experienced SDHT laboratory personnel carried out the Marshall compaction of the research samples, achieving coefficients of variation no higher than seven percent within the volumetric properties of the ten repeat samples of each mix (details shown in Appendix C). The gyratory samples that were compacted by the author, with little laboratory experience, resulted in coefficients of variation for the volumetric properties ranging from 7 to 12 percent within each mix type (see Appendix D). Some of the experimental errors that may have caused these variations include:

- Incorrect aggregate combination within sample.
- Inconsistent sample mixing during the combination of aggregate with asphalt cement for each sample (varied sample mixing time and level of coating of aggregate achieved).
- Variable length of time in oven curing of each sample to reach consistent temperature within the mix prior to compaction.
- Inconsistent scooping of sample into mould resulting in aggregate segregation within the sample.
- Possible cooling of mix during scooping into gyratory mould prior to compaction.

Although the testing equipment and settings were inspected by the author prior to testing, some of the systematic errors that may have affected the sample creation and compaction include:

- Inaccurate scales used to measure the mass of aggregates, asphalt cement, and asphalt mixes.
- Inaccurate temperature of oven used to store and heat aggregate and asphalt samples to the working temperatures required.
- Mechanical settings within the compaction equipment (weight of Marshall hammer and the counter of compaction blows applied, angle of gyration within the gyratory machine, level and uniformity of the compaction pressure applied).

3.4 Chapter Summary

This chapter presented the results of conventional asphalt mix tests completed on the aggregate as well as on Marshall and gyratory-compacted samples for the four research mixes. Physical aggregate properties such as gradation, fracture, fine aggregate angularity, sand equivalent and others were presented. The volumetric properties of Marshall and gyratory samples were examined, and compared to SDHT and

Superpave™ specifications. Results of Marshall stability and flow testing were also presented and discussed.

The particle sizes of the Type 72 mix and the Type 70 mix used in this study varied mainly on the coarse end of the gradation (greater than 5 mm). Fine aggregate angularity for the Type 72 mix ranged from 41.9 percent to 45.1 percent, and increased as the amount of manufactured fine aggregate was increased in the Type 72 mix. Type 70 mix had a fine aggregate angularity of 42.2 percent. The amount of coarse fracture in the Type 72 mixes ranged from 95.2 to 97.8 percent, increasing as the amount of manufactured fine aggregate was increased. The coarse fracture of the Type 70 mix was 90.7 percent.

The average VTM for the 75-blow Marshall compacted samples met SDHT design criteria of three to five percent VTM for each of the research mixes (ranging from 3.9 to 4.3 percent), and resulted in mean VTM close to the Superpave™ recommended target of four percent. In terms of differences in void properties between the four research mixes, the Type 72 mix with 20 percent manufactured fines had the highest VTM with a mean of 4.2 percent, while the Type 70 mix had the lowest VTM, with a mean of 3.9 percent.

Although the gyratory compacted samples on average met the SDHT design criteria for VTM between three and five percent at the N_{design} level of compaction (with mean VTM ranging from 3.1 to 3.6 percent), on average the gyratory samples for each of the research mixes compacted to below the targeted design level of four percent VTM suggested by the Superpave™ mix design process. In terms of differences in gyratory VTM at N_{design} between the four research mixes, the only significant difference noted is between the Type 70 mix (mean VTM of 3.1 percent) and the Type 72 mix with 40 percent manufactured fines (mean VTM of 3.6 percent).

Increasing the amount of manufactured fines in the Type 72 mix resulted in progressively less densification in the mix. The $N_{initial}$ specification was not met by the Type 72 mix with 20 and 40 percent manufactured fines, with mean percent G_{mm} at $N_{initial}$ of 92.3 percent and 89.3 percent, respectively. The Type 72 mix passed the

Superpave^{TM} specification of less than 89 percent G_{mm} at $N_{initial}$ when the manufactured fines content was increased to 60 percent of total fines, based on the average of ten repeat samples, with a mean percent G_{mm} of 88.9 percent. However, even this mix had samples which failed the $N_{initial}$ specification, based on the error bars, which represent two standard deviations (mean – 2 std dev = 88 percent). The Type 70 mix had a mean percent G_{mm} of 90.1 percent, and did not meet the $N_{initial}$ densification criterion.

All of the research mixes compacted to higher percent G_{mm} at N_{design} than 96 percent, therefore resulting in average VTM slightly lower than the desirable four percent that is targeted by Superpave™ (mean VTM ranging from 3.1 to 3.6 percent across mix type). Also, at $N_{maximum}$ the gyratory samples compacted below the SDHT acceptable VTM level of three percent (mean percent G_{mm} ranging from 97.4 to 97.8 percent). This difference in the level of compaction between the Marshall and gyratory methods was expected, since it is generally accepted that the gyratory compaction protocol results in a higher level of compaction.

Marshall stability increased significantly as the proportion of manufactured fines was increased from 20 to 40 (increase of 22 percent), and to 60 percent (increase of 36 percent) in the Type 72 mix, with mean Marshall stabilities of 8,244 N, 10,069 N, 10,084 N, respectively. Further, Marshall stability results for mix Type 70, which has 38 percent of manufactured fines, were statistically the same as those for the Type 72 mix with 40 percent manufactured fines (10,069 N and 10,084 N, respectively), highlighting the dominating effects of manufactured fine aggregate shape, angularity and texture in the dense-graded mix structure.

The mean values of Marshall flow for the four research mixes, although close to the lower acceptable limit of 1.5 mm, met the SDHT design criteria, with mean Marshall flow results for each of the research mixes ranging from 1.8 mm to 2.3 mm. However, there is a significant amount of variability in the test results, especially for the Type 72 mix with 20 percent manufactured fines (CV of 18 percent), and the Type 70 mix (CV of 16 percent), as is indicated by the fact that the lower error bar of two standard deviations results in the mixes failing the minimum SDHT criterion (with mean - 2 std dev = 1.1 mm, and 1.3 mm, respectively). It is also apparent that there is an increasing trend in flow with increasing amounts of manufactured fines. Marshall flow for the Type 72 mix with 60 percent manufactured fines was 28 percent higher than for the Type 72 mix with 20 percent manufactured fines, with a mean of 2.3 mm.

Based on VTM in the Marshall samples, a minimum level of confidence of 98 percent was achieved with ten repeat samples. In the gyratory samples, the author was able to achieve a minimum level of confidence of 85 percent (based on the VTM at N_{design}). The fact that the volumetric properties of the Type 72 mix with 40 percent manufactured fines resulted in a level of confidence significantly lower than that of the other three mixes compacted by the author suggests inconsistencies in the gyratory compaction process by the author across the research mixes.

CHAPTER 4 MECHANISTIC CHARACTERIZATION OF RESEARCH MIXES

The second phase of determining the influence of manufactured fines on Saskatchewan dense graded mixes involved characterizing their behaviour under dynamic loading in various stress state conditions. This was accomplished with the use of the triaxial frequency sweep testing apparatus available at the University of Saskatchewan. This apparatus was first used in 1996, during the design and construction of Saskatchewan's SHRP Specific Pavement Studies – 9A (SPS-9A) asphalt concrete pavement test sections on Highway 16, near Radisson (Berthelot 1999, Czarnecki *et al.* 1999, Anthony and Berthelot 2003). Although this type of testing is not yet widely implemented as part of conventional testing of bituminous materials, its benefits of quantifying fundamental mechanical properties of materials in response to dynamic loading are gaining understanding in the pavement engineering community (NCHRP 2004, NCHRP 2005).

This chapter contains a presentation and discussion of the results of triaxial frequency testing carried out on ten repeat samples for each of the four research mixes. The samples were created using gyratory compaction equipment, and their volumetric properties have been presented and discussed in Chapter Three. All charts illustrate the mean values of ten repeat samples tested, with the error bars representing ± two standard deviations from the mean.

4.1 Triaxial Frequency Sweep Testing Protocol

The triaxial frequency sweep testing was carried out using the Rapid Triaxial Tester (RaTT cell). The RaTT cell has been proven to provide reliable information on mechanistic material properties related to response to dynamic loading (Berthelot *et al.* 1999, Berthelot 1999, Crockford *et al.* 2002, Berthelot *et al.* 2003). This apparatus has

Figure 4.1 University of Saskatchewan Triaxial Frequency Sweep Equipment

been successfully implemented for various research projects related to asphalt concrete mixes (Berthelot 1999, Carlberg 2002, Baumgartner 2005). Further, the RaTT cell is not only limited to testing asphalt mixes - it is capable of testing various road materials, and has been successfully used for this purpose (Berthelot and Gerbrandt 2002, Berthelot *et al.* 2005, Berthelot *et al.* 2007). Figure 4.1 is a photograph of the machine, which is available at the University of Saskatchewan. The machine is capable of testing specimens compacted by the SHRP gyratory compactor, which are of 150 mm in diameter, with a height of 150 ± 5 mm. The large sample size is an advantage over other similar tests, because it helps to eliminate the significance of the disparities present in asphalt mix samples. The disparities within asphalt mixes are inherent because HMAC is a particulate composite of varying aggregate sizes, air voids, and asphalt cement. (Weissman *et al.* 1999).

Using the gyratory compacted samples ensures timely testing, and eliminates the need for coring to obtain four inch specimens, as is necessary for testing in other triaxial apparatus. In addition to requiring more preparation time, coring may introduce irregularities and damage in the form of micro-fracture in the sample.

The RaTT cell features independent closed-loop feedback control of the vertical and confining stresses exerted on the gyratory compacted samples of 150 mm height. The sample is inserted into a rubber membrane, which is used to create radial confinement pneumatically. Sinusoidal axial loading is applied at a specified frequency, and the resulting strains on the sample are measured by two axial and four radially located linear variable differential transducers (LVDT) (Berthelot 1999).

The testing protocols employed in this research study have been developed over the recent years by applying the RaTT cell for various research purposes (Berthelot 1999, Carlberg 2002, Baumgartner 2005). The testing framework was selected in order to investigate the influence of manufactured fines in Saskatchewan mixes subjected to varying load parameters on the following mechanistic properties, which are explained in more detail in Chapter Two, Section 2.7.2:

- Dynamic Modulus, E_d
- Poisson's Ratio, ν
- Recoverable Axial Microstrain, ε_{11}
- Recoverable Radial Microstrain $\varepsilon_{22} = \varepsilon_{33}$
- Phase Angle, δ

The RaTT cell is capable of testing materials by varying the following parameters, whose influence on the response of the research mixes was also investigated:

- Magnitude of axial load application (simulates varying vehicle loadings).
- Frequency of axial load application (simulates varying traffic speeds).
- Magnitude of radial confinement (simulates various locations within a pavement structure).
- Testing temperature (simulates varying atmospheric conditions).

In order to subject the samples to uniform loading conditions, and to be able to best quantify the influence of the changes in the various condition states simulated during testing, all samples were subjected to the same testing sequence, beginning with the least damage-causing, and progressively applying increasingly damaging condition states, as shown in Table 4.1. As can be seen, the peak axial traction was maintained at a constant of 600 kPa, and the peak radial traction was varied, in order to simulate various deviatoric stress levels. The samples were conditioned to 20°C and tested as outlined in Table 4.1.

Table 4.1 Triaxial Frequency Sweep Testing Sequence

Testing Sequence	Peak Axial Traction kPa	Peak Radial Traction kPa	Deviatoric Stress σ_D kPa	Axial Load Frequency Hz
1	600	230	370	10
2	600	230	370	5
3	600	230	370	1
4	600	230	370	0.5
5	600	175	425	10
6	600	175	425	5
7	600	175	425	1
8	600	175	425	0.5
9	600	100	500	10
10	600	100	500	5
11	600	100	500	1
12	600	100	500	0.5

The following sections contain a discussion of the test results with respect to the influence of manufactured fines on the mechanistic properties of the research mixes by analysing the results for the Type 72 mix with varying levels of manufactured fines content, as well as the influence of the aggregate skeleton, which is accomplished by comparing the results for the Type 72 mixes with those for the Type 70 mix with 38 percent manufactured fines.

All relationships are examined by presenting the results at various stress states, at a testing temperature of 20°C and two loading frequencies. The highest loading frequency of 10 Hz was selected because it is intended to simulate highway traffic speeds, and the lowest testing frequency of 0.5 Hz was selected because it simulates loading in slow moving traffic conditions. The Type 72 mix with 20 percent manufactured fines was used as a baseline for quantifying the magnitude of changes in the test results between the four research mixes. Statistical significance of the results was investigated by performing analysis of variance on the influence of mix type and stress state on each dependent variable, and by applying Tukey's pairwise comparison for a more detailed analysis.

4.2 Dynamic Modulus Characterization of Research Mixes

As explained in Chapter Two, the dynamic modulus, E_d, is a measure of stiffness represented in the RaTT cell by the absolute value of peak stress to peak strain during material testing under specified test conditions. Dynamic modulus is used to quantify the stress-strain relationships in a pavement structure under an applied load. A higher stiffness modulus indicates that a given applied stress results in lower strain in the mixture. The influence of the manufactured fines content and the change in aggregate skeleton on the dynamic modulus measured at a temperature of 20°C and loading frequencies of 10 Hz and 0.5 Hz are discussed below.

Table 4.2 and Figure 4.2 show the dynamic modulus of the research mixes at a frequency of 10 Hz, across the three levels of applied deviatoric stress. Table 4.3 and Figure 4.3 show the dynamic modulus of the research mixes at a frequency of 0.5 Hz. The lower loading frequency results in significantly lower magnitudes of the dynamic modulus obtained from the applied loading, as has also been shown in previous triaxial frequency sweep testing (Carlberg 2003, Baumgartner 2005). In fact, the modulus decreases by approximately half the magnitude when frequency is reduced from 10 to 0.5 Hz. Also, as deviatoric stress increases, dynamic modulus decreases.

Using the Type 72 mix with 20 percent of manufactured fines as a baseline, it can be seen that at 10 Hz there is minimal change in the dynamic modulus between 20 and 40 percent of manufactured fines content (one percent increase); however, there is a significant increase once the aggregate skeleton contains 60 percent manufactured fines. In fact, the dynamic modulus increases by approximately 50 percent when the aggregate skeleton is modified from 20 to 60 percent manufactured fines, at each stress state, as is shown in Table 4.2. Similarly, at the frequency of 0.5 Hz there is minimal change in the

Table 4.2 Mean Dynamic Modulus across Stress State at 10 Hz and 20°C

Deviatoric Stress, σ_D (kPa)	Mix Type	Mean Dynamic Modulus, E_d (MPa)	Coefficient of Variation (%)	% Difference from T72(20%MF)
370	T72(20%MF)	2167	3	---
	T72(40%MF)	2193	4	1%
	T72(60%MF)	3292	15	52%
	T70(38%MF)	2317	13	7%
425	T72(20%MF)	1963	2	---
	T72(40%MF)	1987	5	1%
	T72(60%MF)	2967	16	51%
	T70(38%MF)	2085	11	6%
500	T72(20%MF)	1832	3	---
	T72(40%MF)	1831	3	0%
	T72(60%MF)	2784	18	52%
	T70(38%MF)	1950	10	6%

Table 4.3 Mean Dynamic Modulus across Stress State at 0.5 Hz and 20°C

Deviatoric Stress σ_D (kPa)	Mix Type	Mean Dynamic Modulus, E_d (MPa)	Coefficient of Variation (%)	% Difference from T72(20%MF)
370	T72(20%MF)	1193	3	---
	T72(40%MF)	1173	3	-2%
	T72(60%MF)	1489	15	25%
	T70(38%MF)	1200	8	1%
425	T72(20%MF)	1017	3	---
	T72(40%MF)	1020	7	0.3%
	T72(60%MF)	1265	15	24%
	T70(38%MF)	1030	7	1%
500	T72(20%MF)	914	3	---
	T72(40%MF)	907	3	-1%
	T72(60%MF)	1159	16	27%
	T70(38%MF)	958	6	5%

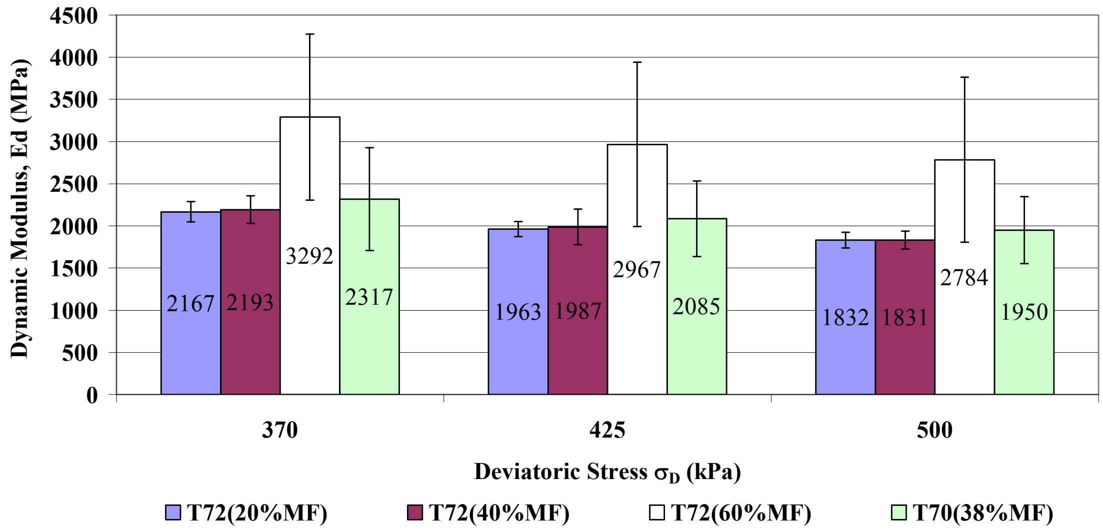

Figure 4.2 Mean Dynamic Modulus across Stress State at 10 Hz and 20°C (± 2 SD)

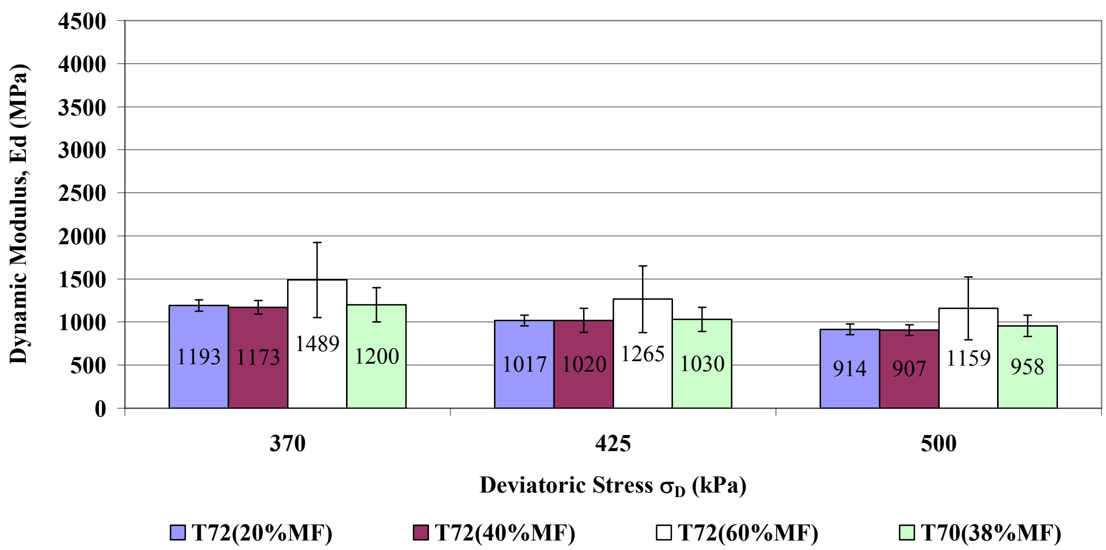

Figure 4.3 Mean Dynamic Modulus across Stress State at 0.5 Hz and 20°C (± 2 SD)

dynamic modulus between 20 and 40 percent of manufactured fines content; however, there is a significant increase for the Type 72 mix with 60 percent manufactured fines evident at each stress state. Although the increase is not as large as with the higher testing frequency (50 percent at 10 Hz vs. 25 percent at 0.5 Hz), the dynamic modulus increases by approximately 25 percent when the aggregate skeleton is modified from 20 to 60 percent manufactured fines, as is illustrated in Table 4.3.

It is also evident that there is minimal difference between the mean dynamic modulus of the Type 70 mix, which SDHT considers a structural mix, and the Type 72 mixes with 20 and 40 percent manufactured fines content, across deviatoric stress and at both loading frequencies tested. In fact, the mean E_d of the Type 70 mix is only approximately 6 percent higher than that for the Type 72 mix with 20 percent manufactured fines at the frequency of 10 Hz, and at the frequency of 0.5 Hz, the mean dynamic modulus of the Type 70 mix is only approximately one percent higher than that for the Type 72 mix with 20 percent manufactured fines. This further confirms the observations made based on Marshall stability testing, which are discussed in the previous Chapter.

Keeping in mind that the error bars in Figure 4.2 and Figure 4.3 represent ± two standard deviations, and inspecting the coefficients of variation listed in Table 4.2 and Table 4.3, it is clear that there is a high variability within the ten repeat samples for the Type 72 mix with 60 percent manufactured fines (CV ranging from 15 to 18 percent). This may be attributed to the fact that increasing the manufactured fines content, and therefore the total content of manufactured materials, results in more variability in the particle arrangement during compaction, therefore increasing the variability in the response to axial loading.

Analysis of variance of the results at each loading frequency illustrates the fact that the type of mix is significant in the dynamic modulus results (Table 4.4 and Table 4.5). Tukey's Pairwise Comparison shown in Table 4.6 and Table 4.7 was performed to further investigate the relationship between the mixes at various stress states applied. Tukey's analysis compares the mean of each population against the mean of each of the other populations, creating separate groups for results that are statistically different.

Table 4.4 Analysis of Variance for Dynamic Modulus at 10 Hz and 20°C

Effect	Sum of Squares	Degrees of Freedom	Mean Squares	F-Test Statistic	P-value
Mix Type	*21829940*	*3*	*7276647*	*93.78*	*0.00*
Deviatoric Stress	*3140620*	*2*	*1570310*	*20.24*	*0.00*
Mix Type*Deviatoric Stress	98941	6	16490	0.21	0.97
Error	8379932	108	77592		

Table 4.5 Analysis of Variance for Dynamic Modulus at 0.5 Hz and 20°C

Effect	Sum of Squares	Degrees of Freedom	Mean Squares	F-Test Statistic	P-value
Mix Type	*1516876*	*3*	*505625*	*41.16*	*0.00*
Deviatoric Stress	*1605018*	*2*	*802509*	*65.33*	*0.00*
Mix Type * Deviatoric Stress	25465	6	4244	0.35	0.91
Error	1326606	108	12283		

As can be seen in Table 4.6, the Type 72 mix with 60 percent manufactured fines results in significantly higher dynamic modulus across the three stress states than any of the other three mixes at 10 Hz. Also, the dynamic modulus for the Type 70 mix does not perform different from the Type 72 mixes with 20 and 40 percent manufactured fines. Tukey's Pairwise Comparison shown in Table 4.7 shows that the Type 72 mix with 60 percent manufactured fines results in significantly higher dynamic modulus across the three stress states than any of the other three mixes at 0.5 Hz. Also, the dynamic modulus for the Type 70 mix does not perform different from the Type 72 mixes with 20 and 40 percent manufactured fines. In addition, stress state also significantly impacts the results at both frequencies. Namely, as deviatoric stress increases, dynamic modulus decreases.

Table 4.6 Tukey's Homogeneous Groups for Dynamic Modulus at 10 Hz and 20°C

Deviatoric Stress σ_D (kPa)	Mix Type	Mean Dynamic Modulus E_d (MPa)	Tukey's Homogeneous Groups			
			A	B	C	D
370	T72 (20%MF)	2167	****	****		
	T72 (40%MF)	2193	****	****		
	T70 (38%MF)	2317		****		
	T72 (60%MF)	3292				****
425	T72 (20%MF)	1963	****	****		
	T72 (40%MF)	1987	****	****		
	T70 (38%MF)	2085	****	****		
	T72 (60%MF)	2967			****	****
500	T72 (20%MF)	1832	****			
	T72 (40%MF)	1831	****			
	T70 (38%MF)	1950	****	****		
	T72 (60%MF)	2784			****	

Table 4.7 Tukey's Homogeneous Groups for Dynamic Modulus at 0.5 Hz and 20°C

Deviatoric Stress σ_D (kPa)	Mix Type	Mean Dynamic Modulus E_d (MPa	Tukey's Homogeneous Groups				
			A	B	C	D	E
370	T72(40%MF)	1173		****	****	****	
	T72(20%MF)	1193			****	****	
	T70(38%MF)	1200				****	
	T72(60%MF)	1489					****
425	T72(20%MF)	1017	****	****			
	T72(40%MF)	1020	****	****			
	T70(38%MF)	1030	****	****	****		
	T72(60%MF)	1265				****	
500	T72(60%MF)	1159		****	****	****	
	T72(40%MF)	907	****				
	T72(20%MF)	914	****				
	T70(38%MF)	958	****				

4.3 Recoverable Axial Microstrain Characterization of Research Mixes

The amount of recoverable axial microstrain (RAMS) is a measure of the recoverable portion of the strain resulting from the dynamic loading in the RaTT cell along the same vertical axis on which the loading is applied.

Table 4.8 and Figure 4.4 illustrate the average amount of RAMS of ten repeat samples across mix type, for each level of deviatoric stress applied, at a loading frequency of 10 Hz. Table 4.9 and Figure 4.5 show the average RAMS across mix type and deviatoric stress at a loading frequency of 0.5 Hz. As can be seen when comparing the two sets of data at different frequencies, there is a significant change in the overall magnitude of the microstrains between 10 Hz and 0.5 Hz. As can be expected, the slower loading frequency of 0.5 Hz results in almost twice the amount of strain than the frequency of 10 Hz. Also, an increase in deviatoric stress results in an increase in recoverable axial microstrains.

Using the Type 72 mix with 20 percent of manufactured fines as a baseline, it can be seen in Table 4.8 that at 10 Hz there is minimal change in the mean RAMS between 20 and 40 percent of manufactured fines content, however, there is a decrease in the order of 30 percent in the mean RAMS, at each stress state, once the aggregate skeleton contains 60 percent manufactured fines. Similarly, at the frequency of 0.5 Hz there is minimal change in the average RAMS between 20 and 40 percent of manufactured fines content, however, there is a significant increase for the Type 72 mix with 60 percent manufactured fines. Although the increase is not as large as with the higher testing frequency (30 percent vs. 18 percent), as is illustrated in Table 4.9. There appears to be a slight decrease in RAMS for the Type 70 mix at 10 Hz, in the order of 5 percent.

There is high variability within the RAMS for the Type 72 mix with 60 percent manufactured fines (CV ranging from 13 to 18 percent). It is suspected that this variability exists for similar reasons named for the dynamic modulus. Namely, it may be attributed to the fact that increasing the manufactured fines content results in more variability in the particle arrangement due to aggregate shape, therefore increasing the variability in the response to axial loading.

Table 4.8 Mean Recoverable Axial Microstrain across Stress State at 10 Hz and 20°C

Deviatoric Stress, σ_D (kPa)	Mix Type	Mean Recoverable Axial Microstrain (10^{-6})	Coefficient of Variation (%)	% Difference from T72(20%MF)
370	T72(20%MF)	269	3	---
	T72(40%MF)	267	4	-1%
	T72(60%MF)	180	15	-33%
	T70(38%MF)	255	10	-5%
425	T72(20%MF)	296	2	---
	T72(40%MF)	293	5	-1%
	T72(60%MF)	200	16	-32%
	T70(38%MF)	281	9	-5%
500	T72(20%MF)	317	3	---
	T72(40%MF)	317	3	0%
	T72(60%MF)	214	18	-32%
	T70(38%MF)	300	9	-5%

Table 4.9 Mean Recoverable Axial Microstrain across Stress State at 0.5 Hz and 20°C

Deviatoric Stress σ_D (kPa)	Mix Type	Mean Recoverable Axial Microstrain (10^{-6})	Coefficient of Variation (%)	% Difference from T72(20%MF)
370	T72(20%MF)	502	3	---
	T72(40%MF)	511	3	2%
	T72(60%MF)	409	12	-19%
	T70(38%MF)	501	7	0%
425	T72(20%MF)	587	3	---
	T72(40%MF)	588	6	0.1%
	T72(60%MF)	480	13	-18%
	T70(38%MF)	581	6	-1%
500	T72(20%MF)	653	3	---
	T72(40%MF)	658	3	1%
	T72(60%MF)	524	13	-20%
	T70(38%MF)	625	6	-4%

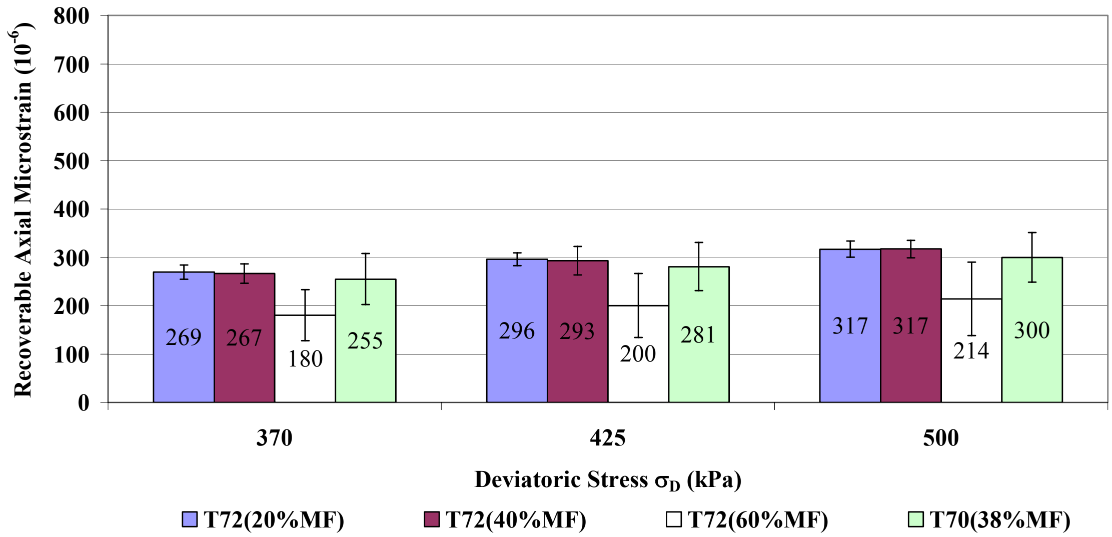

Figure 4.4 Mean Recoverable Axial Microstrain across Stress State at 10 Hz and 20°C (± 2 SD)

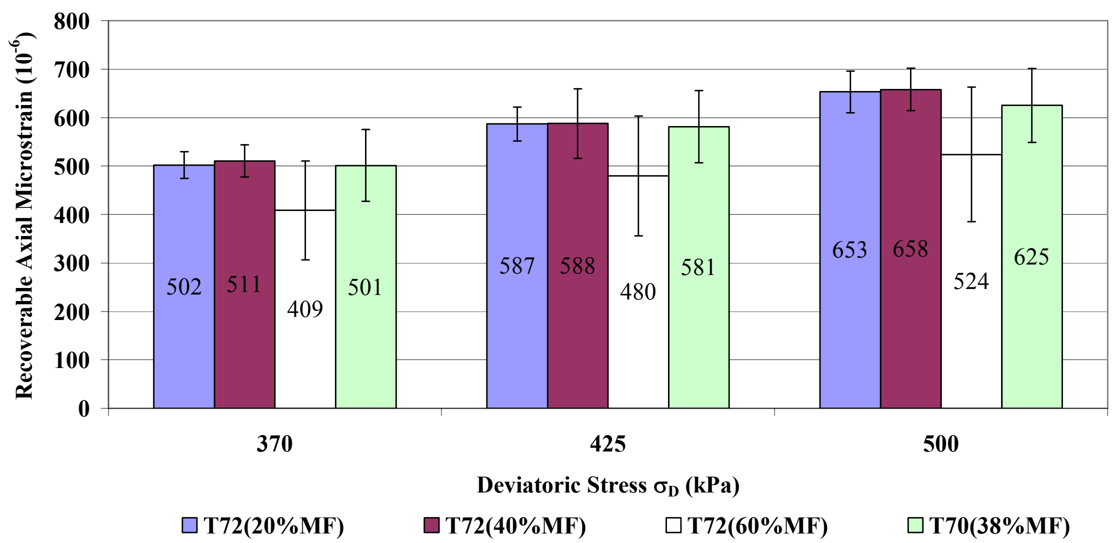

Figure 4.5 Mean Recoverable Axial Microstrain across Stress State at 0.5 Hz and 20°C (± 2 SD)

Analysis of variance of the results at each loading frequency illustrates that the type of mix as well as stress state are significant in the RAMS results (Table 4.4 and Table 4.5). Further statistical analysis using Tukey's homogeneous groups clearly identifies the Type 72 mix with 60 percent manufactured fines as resulting in the lowest recoverable axial microstrains when compared to the other three mixes at each stress state and frequency tested (as shown in Table 4.12 and Table 4.13). It is evident at both frequencies and all stress states that Type 70 RAMS are statistically not significantly different than the Type 72 mixes with 20 and 40 percent manufactured fines, respectively. Also, an increase in deviatoric stress results in increased recoverable axial microstrains, as would be expected.

Table 4.10 Analysis of Variance for Recoverable Axial Microstrain at 10 Hz and 20°C

Effect	Sum of Squares	Degrees of Freedom	Mean Squares	F-Test Statistic	P-value
Mix Type	*186614*	*3*	*62205*	*129.70*	*0.00*
Deviatoric Stress	*39219*	*2*	*19609*	*40.89*	*0.00*
Mix Type * Deviatoric Stress	821	6	137	0.29	0.94
Error	51798	108	480		

Table 4.11 Analysis of Variance for Recoverable Axial Microstrain at 0.5 Hz and 20°C

Effect	Sum of Squares	Degrees of Freedom	Mean Squares	F-Test Statistic	P-value
Mix Type	*264716*	*3*	*88239*	*57.49*	*0.00*
Deviatoric Stress	*365152*	*2*	*182576*	*118.94*	*0.00*
Mix Type * Deviatoric Stress	5414	6	902	0.59	0.74
Error	165776	108	1535		

Table 4.12 Tukey's Homogeneous Groups for Recoverable Axial Microstrain at 10 Hz and 20°C

Deviatoric Stress σ_D (kPa)	Mix Type	Mean Recoverable Axial Microstrain (10^{-6})	Tukey's Homogeneous Groups					
			A	B	C	D	E	F
370	T72(60%MF)	180	****					
	T70(38%MF)	255			****			
	T72(20%MF)	269			****	****	****	
	T72(40%MF)	267			****	****		
425	T72(60%MF)	200	****	****				
	T70(38%MF)	281			****	****	****	
	T72(40%MF)	293				****	****	****
	T72(20%MF)	296				****	****	****
500	T72(60%MF)	214		****				
	T70(38%MF)	300					****	****
	T72(20%MF)	317						****
	T72(40%MF)	317						****

Table 4.13 Tukey's Homogeneous Groups for Recoverable Axial Microstrain at 0.5 Hz and 20°C

Deviatoric Stress σ_D (kPa)	Mix Type	Mean Recoverable Axial Microstrain (10^{-6})	Tukey's Homogeneous Groups				
			A	B	C	D	E
370	T72(60%MF)	409	****				
	T70(38%MF)	501		****			
	T72(20%MF)	502		****			
	T72(40%MF)	511		****			
425	T72(60%MF)	480		****			
	T70(38%MF)	581			****	****	
	T72(20%MF)	587				****	
	T72(40%MF)	588				****	
500	T72(60%MF)	524		****	****		
	T70(38%MF)	625				****	****
	T72(20%MF)	653					****
	T72(40%MF)	658					****

4.4 Recoverable Radial Microstrain Characterization of Research Mixes

The amount of recoverable radial microstrain (RRMS) is a measure of the recoverable portion of the strain resulting from the dynamic loading in the RaTT cell along a horizontal axis at mid-height of the sample, which is perpendicular to the direction of the dynamic loading applied. RRMS is an indicator of shear strength of the material.

Table 4.14 and Figure 4.6 illustrates the average amount of RRMS of ten repeat samples across mix type, for each level of deviatoric stress applied, at a loading frequency of 10 Hz. Table 4.15 and Figure 4.7 show the average RRMS across mix type and deviatoric stress at a loading frequency of 0.5 Hz. The RRMS more than double in magnitude when the frequency of loading is reduced from 10 Hz to 0.5 Hz, indicating a significant increase in shear forces with the lowered loading frequency. Also, the RRMS increase as deviatoric stress is increased.

Using the Type 72 mix with 20 percent of manufactured fines as a baseline, it can be seen in Table 4.14 that at 10 Hz there is minimal change in the RRMS between 20 and 40 percent of manufactured fines content, however, there is a decrease in the order of 18 to 22 percent in the mean RRMS, depending on stress state, once the aggregate skeleton contains 60 percent manufactured fines. Similarly, at the frequency of 0.5 Hz there is minimal change in the RAMS between 20 and 40 percent of manufactured fines content, however, the mean RRMS for the Type 72 mix with 60 percent manufactured fines decreases by a minimum of 7 percent, depending on the stress state. The Type 70 mix results in slightly lower mean RRMS than the baseline mix at 10 Hz (by approximately 7 percent), however, at 0.5 Hz there appears to be minimal difference.

There is increased variability in RRMS within each mix type for the Type 72 mix with 60 percent manufactured fines (CV ranging from 17 to 20 percent). It is suspected that this variability exists for similar reasons listed when discussing the variability present in the dynamic modulus and RAMS for the Type 72 mix with 60 percent manufactured fines. The Type 70 mix is also showing increased variability in RRMS (CV ranging from 9 to 17 percent).

Table 4.14 Mean Recoverable Radial Microstrain across Stress State at 10 Hz and 20°C

Deviatoric Stress, σ_D (kPa)	Mix Type	Mean Recoverable Radial Microstrain (10^{-6})	Coefficient of Variation (%)	% Difference from T72(20%MF)
370	T72(20%MF)	88	7	---
	T72(40%MF)	87	5	-1%
	T72(60%MF)	68	17	-22%
	T70(38%MF)	82	14	-7%
425	T72(20%MF)	93	8	---
	T72(40%MF)	90	8	-3%
	T72(60%MF)	75	17	-20%
	T70(38%MF)	87	16	-6%
500	T72(20%MF)	93	9	---
	T72(40%MF)	94	11	2%
	T72(60%MF)	76	20	-18%
	T70(38%MF)	86	17	-8%

Table 4.15 Mean Recoverable Radial Microstrain across Stress State at 0.5 Hz and 20°C

Deviatoric Stress σ_D (kPa)	Mix Type	Mean Recoverable Radial Microstrain (10^{-6})	Coefficient of Variation (%)	% Difference from T72(20%MF)
370	T72(20%MF)	216	4	---
	T72(40%MF)	222	6	3%
	T72(60%MF)	200	18	-7%
	T70(38%MF)	218	9	1%
425	T72(20%MF)	260	5	---
	T72(40%MF)	263	7	1.2%
	T72(60%MF)	232	20	-11%
	T70(38%MF)	262	11	1%
500	T72(20%MF)	292	6	---
	T72(40%MF)	293	5	0%
	T72(60%MF)	250	19	-14%
	T70(38%MF)	279	11	-5%

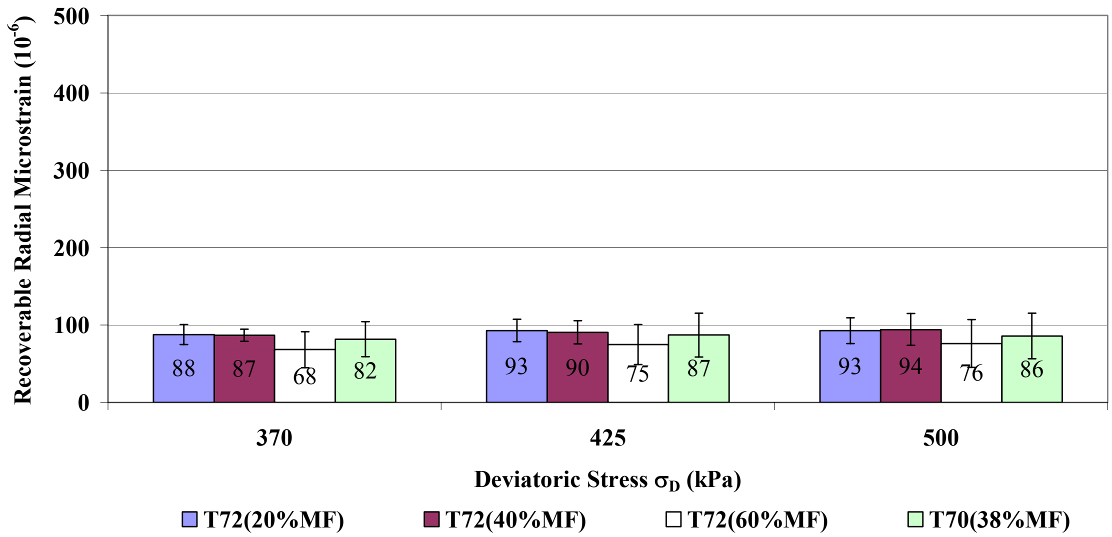

Figure 4.6 Mean Recoverable Radial Microstrain across Stress State at 10 Hz and 20°C (± 2 SD)

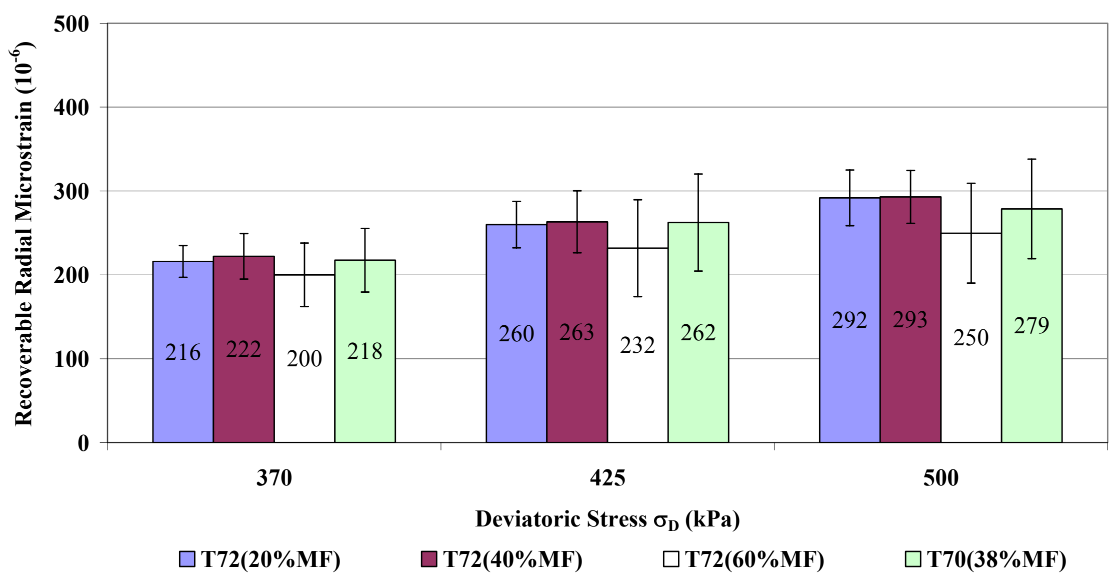

Figure 4.7 Mean Recoverable Radial Microstrain across Stress State at 0.5 Hz and 20°C (± 2 SD)

Table 4.16 Analysis of Variance for Recoverable Radial Microstrain at 10 Hz and 20°C

Effect	Sum of Squares	Degrees of Freedom	Mean Squares	F-Test Statistic	P-value
Mix Type	*6388*	*3*	*2129*	*17.92*	*0.00*
Deviatoric Stress	*888*	*2*	*444*	*3.73*	*0.027*
Mix Type * Deviatoric Stress	95.3	6	16	0.13	0.99
Error	12828	108	119		

Table 4.17 Analysis of Variance for Recoverable Radial Microstrain at 0.5 Hz and 20°C

Effect	Sum of Squares	Degrees of Freedom	Mean Squares	F-Test Statistic	P-value
Mix Type	*19431*	*3*	*6477*	*8.53*	*3.9E-05*
Deviatoric Stress	*84684*	*2*	*42342*	*55.73*	*0.00*
Mix Type * Deviatoric Stress	2348	6	391	0.52	0.80
Error	82045	108	760		

By examining the ANOVA results show in Table 4.16 and Table 4.17, it is apparent by the increased F-test statistic that at the frequency of 0.5 Hz the effects of stress state are magnified, reducing the significance of the influence from mix type.

Table 4.18 shows the results of the pairwise comparison using Tukey's method of homogeneous groups for the RRMS results across mix types at the three deviatoric stress states, at a frequency of 10 Hz. Table 4.19 shows the Tukey's homogeneous groups at 0.5 Hz. At 10 Hz the RRMS for the Type 72 mix with 60 percent manufactured fines are significantly lower from the other two Type 72 mixes at each deviatoric stress level.

Table 4.18 Tukey's Homogeneous Groups for Recoverable Radial Microstrain at 10 Hz and 20°C

Deviatoric Stress σ_D (kPa)	Mix Type	Mean Recoverable Radial Microstrain (10^{-6})	Tukey's Homogeneous Groups		
			A	B	C
370	T72(60%MF)	68	****		
	T70(38%MF)	82	****	****	****
	T72(40%MF)	87		****	****
	T72(20%MF)	88		****	****
425	T72(60%MF)	75	****	****	
	T70(38%MF)	87		****	****
	T72(40%MF)	90		****	****
	T72(20%MF)	93			****
500	T72(60%MF)	76	****	****	
	T70(38%MF)	86		****	****
	T72(20%MF)	93			****
	T72(40%MF)	94			****

Table 4.19 Tukey's Homogeneous Groups for Recoverable Radial Microstrain at 0.5 Hz and 20°C

Deviatoric Stress σ_D (kPa)	Mix Type	Mean Recoverable Radial Microstrain (10^{-6})	Tukey's Homogeneous Groups				
			A	B	C	D	E
370	T72(60%MF)	200	****				
	T72(20%MF)	216	****	****			
	T70(38%MF)	218	****	****			
	T72(40%MF)	222	****	****	****		
425	T72(60%MF)	232	****	****	****		
	T72(20%MF)	260			****	****	****
	T70(38%MF)	262			****	****	****
	T72(40%MF)	263			****	****	****
500	T72(60%MF)	250		****	****	****	
	T70(38%MF)	279				****	****
	T72(20%MF)	292					****
	T72(40%MF)	293					****

At 0.5 Hz the effects of the reduced frequency and the deviatoric stress levels are dominant, and there is a lot more interaction between the RRMS results of the research mixes. The benefits of increased manufactured fines are only significant when the deviatoric stress is 500 kPa, where the Type 72 mix with 60 percent manufactured fines results in reduced RRMS (250E-6) when compared to the Type 72 mix with 20 and 40 percent manufactured fines (292E-6 and 293E-6, respectively). The RRMS for the Type 70 mix are statistically the same as the RRMS for the Type 72 mix with 20 and 40 percent manufactured fines, respectively, at each stress state and frequency.

4.5 Poisson's Ratio Characterization of Research Mixes

As previously defined in Chapter Two, Poisson's ratio, ν, is the relationship of the lateral strain to the axial strain, and can be obtained from the RaTT cell results by dividing the recoverable radial microstrains by the recoverable axial microstrains.

Table 4.20 and Figure 4.8 show the results of Poisson's ratio across stress states at 10 Hz. Table 4.21 and Figure 4.9 show the Poisson's ratio for the research mixes at 0.5 Hz. There is a substantial difference in magnitude between Poisson's ratio at 10 Hz and at 0.5 Hz. Traditionally, Poisson's ratio for asphalt mixes is assumed to be in the order of 0.35 for the purposes of structural parameters calculations and modeling. It appears that while this estimate is reasonable at high frequencies, such as 10 Hz, at 0.5 Hz the average Poisson's ratios for all the mixes tested in this research are 0.43 or higher.

As can be seen by setting the Type 72 mix with 20 percent as a baseline, there is a significant increase in Poisson's ratio at 10 Hz when the manufactured fines are increased to 60 percent for the Type 72 mix, ranging from 17 to 23 percent higher than the baseline, depending on stress state. Although this same observation can be made at 0.5 Hz, the magnitude of the difference is substantially reduced, to 6 to 13 percent higher than the baseline (see Table 4.21). There is minimal difference between the Type 70 mix and the baseline, in the order of zero to two percent, at each stress state and frequency.

Table 4.20 Mean Poisson's Ratio across Stress State at 10 Hz and 20°C

Deviatoric Stress, σ_D (kPa)	Mix Type	Mean Poisson's Ratio ν	Coefficient of Variation (%)	% Difference from T72(20%MF)
370	T72(20%MF)	0.32	6	---
	T72(40%MF)	0.33	3	0%
	T72(60%MF)	0.38	13	17%
	T70(38%MF)	0.32	14	-1%
425	T72(20%MF)	0.31	6	---
	T72(40%MF)	0.31	6	-1%
	T72(60%MF)	0.38	14	20%
	T70(38%MF)	0.31	15	-1%
500	T72(20%MF)	0.29	7	---
	T72(40%MF)	0.30	11	2%
	T72(60%MF)	0.36	18	23%
	T70(38%MF)	0.29	16	-2%

Table 4.21 Mean Poisson's Ratio across Stress State at 0.5 Hz and 20°C

Deviatoric Stress σ_D (kPa)	Mix Type	Mean Poisson's Ratio ν	Coefficient of Variation (%)	% Difference from T72(20%MF)
370	T72(20%MF)	0.43	3	---
	T72(40%MF)	0.43	4	1%
	T72(60%MF)	0.49	11	13%
	T70(38%MF)	0.43	9	1%
425	T72(20%MF)	0.44	3	---
	T72(40%MF)	0.45	3	1%
	T72(60%MF)	0.48	11	8%
	T70(38%MF)	0.45	9	2%
500	T72(20%MF)	0.45	3	---
	T72(40%MF)	0.45	4	0%
	T72(60%MF)	0.47	10	6%
	T70(38%MF)	0.45	8	0%

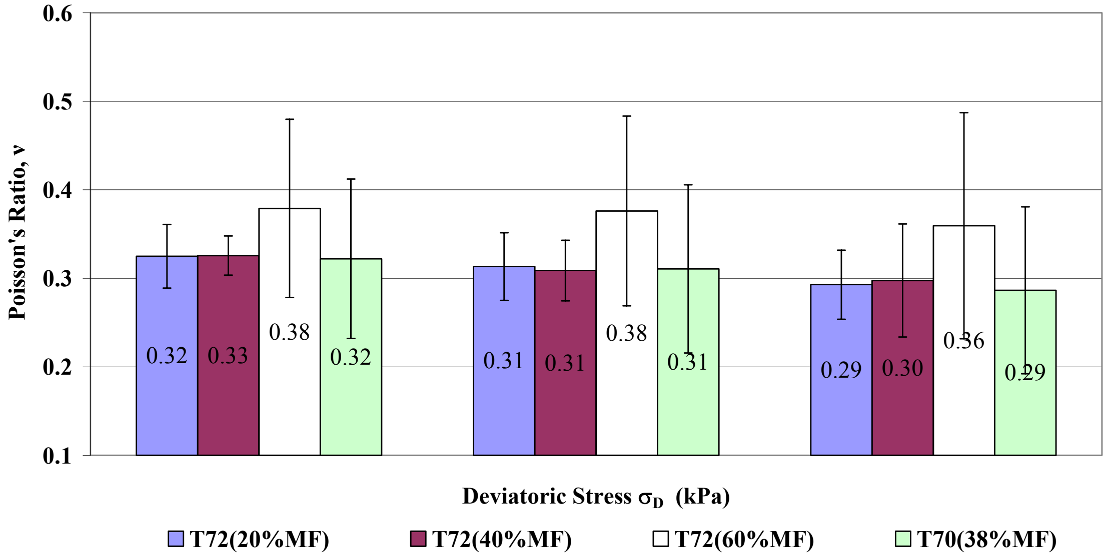

Figure 4.8 Mean Poisson's Ratio across Stress State at 10 Hz and 20°C (± 2 SD)

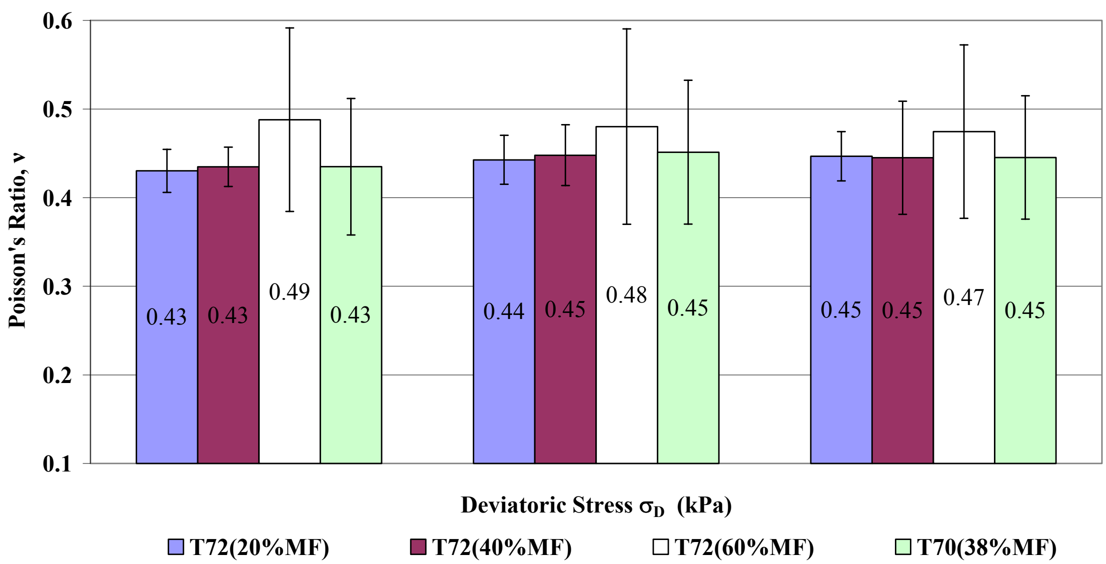

Figure 4.9 Mean Poisson's Ratio across Stress State at 0.5 Hz and 20°C (± 2 SD)

Comparable to the behaviour noted in RAMS, the Type 72 mix with 60 percent manufactured fines and the Type 70 mix both show high variability in the Poisson's ratio results, as is indicated by the large error bars in the illustration charts at each frequency, and the coefficients of variation in Table 4.20 and Table 4.21 (CV ranging from 10 to 18 percent, and from 8 to 16 percent, respectively). This variability can likely be attributed to the aggregate structures being more variable for the mixes containing more fractured aggregate, similar to the variability of the previous variables reviewed.

The ANOVA table for Poisson's ratio at a frequency of 10 Hz indicates that at the high frequency, it is the mix type that has a dominant influence on Poisson's ratio results, with the deviatoric stress also being significant (see Table 4.22). At 0.5 Hz the only significant independent variable is mix type, as can be seen in Table 4.23.

Table 4.23 and Table 4.24 shows the pairwise comparison of Poisson's ratio across stress state, at the loading frequency of 10 Hz. There is a lot of interaction between the results for the different mix types, especially at the lower deviatoric stress state. However, when the deviatoric stress is as high as 425 kPa and 500 kPa, the Type 72 mix with 60 percent manufactured fines has a significantly higher Poisson's ratio than the other three mixes, at 0.38 and 0.36, respectively for each stress state. At the loading frequency of 0.5 Hz, the Type 72 mix has a higher Poisson's ratio (0.49) only at the lowest deviatoric stress level of 370 kPa. As the deviatoric stress increases, with the combined effects of the slower loading, the influence of mix type is no longer significant, as can be seen in Table 4.25. The Type 70 mix resulted in Poisson's ratio statistically the same as the Type 72 mixes with 20 and 40 percent manufactured fines at each stress state and frequency.

Table 4.22 Analysis of Variance for Poisson’s Ratio at 10 Hz and 20°C

Effect	Sum of Squares	Degrees of Freedom	Mean Squares	F-Test Statistic	P-value
Mix Type	*0.088*	*3*	*0.029*	*18.97*	*0.00*
Deviatoric Stress	*0.02*	*2*	*0.009*	*5.54*	*0.005*
Mix Type * Deviatoric Stress	0.00112	6	0.00019	0.12	0.99
Error	0.17	108	0.00154		

Table 4.23 Analysis of Variance for Poisson’s Ratio at 0.5 Hz and 20°C

Effect	Sum of Squares	Degrees of Freedom	Mean Squares	F-Test Statistic	P-value
Mix Type	*0.0341*	*3*	*0.011*	*9.83*	*9.0E-06*
Deviatoric Stress	0.0015	2	0.0008	0.66	0.52
Mix Type * Deviatoric Stress	0.0032	6	0.0005	0.46	0.84
Error	0.1250	108	0.0012		

Table 4.24 Tukey's Homogeneous Groups for Poisson’s Ratio at 10 Hz and 20°C

Deviatoric Stress σ_D (kPa)	Mix Type	Mean Poisson’s Ratio ν	Tukey’s Homogeneous Groups		
			A	B	C
370	T70(38%MF)	0.32	****	****	****
	T72(20%MF)	0.32	****	****	****
	T72(40%MF)	0.33	****	****	****
	T72(60%MF)	0.38			****
425	T72(40%MF)	0.31	****	****	
	T70(38%MF)	0.31	****	****	
	T72(20%MF)	0.31	****	****	
	T72(60%MF)	0.38			****
500	T72(20%MF)	0.29	****		
	T72(40%MF)	0.30	****		
	T70(38%MF)	0.29	****		
	T72(60%MF)	0.36		****	****

Table 4.25 Tukey's Homogeneous Groups for Poisson's Ratio at 0.5 Hz and 20°C

Deviatoric Stress σ_D (kPa)	Mix Type	Mean Poisson's Ratio ν	Tukey's Homogeneous Groups A	Tukey's Homogeneous Groups B
370	T72(20%MF)	0.43	****	
	T72(40%MF)	0.43	****	
	T70(38%MF)	0.43	****	
	T72(60%MF)	0.49		****
425	T72(20%MF)	0.44	****	****
	T72(40%MF)	0.45	****	****
	T70(38%MF)	0.45	****	****
	T72(60%MF)	0.48	****	****
500	T72(40%MF)	0.45	****	****
	T70(38%MF)	0.45	****	****
	T72(20%MF)	0.45	****	****
	T72(60%MF)	0.47	****	****

4.6 Phase Angle Characterization of Research Mixes

Phase angle, δ, is the shift between the applied stress and the resultant strain, and is an indication of the visco-elastic properties of the material tested.

Table 4.26 and Figure 4.10 show the average phase angle at 10 Hz across the three deviatoric stress states for each of the research mixes. Table 4.27 and Figure 4.11 show the average phase angle results at 0.5 Hz across stress state and mix type. The phase angle magnitude decreases as frequency is reduced from 10 to 0.5 Hz.

Using the Type 72 mix with 20 percent manufactured fines as a baseline, it can be seen that at 10 Hz an increase of manufactured fines up to 60 percent of total fines in the Type 72 mix results in a significant increase of phase angle, ranging from 26 to 28 percent, depending on stress state. A similar increase is observed at 0.5 Hz: the phase angle is from 19 to 24 percent higher for the mix with 60 percent manufactured fines than the baseline mix at 0.5 Hz, across deviatoric stress state.

Table 4.26 Mean Phase Angle across Stress State at 10 Hz and 20°C

Deviatoric Stress, σ_D (kPa)	Mix Type	Mean Phase Angle δ (°)	Coefficient of Variation (%)	% Difference from T72(20%MF)
370	T72(20%MF)	20.5	3	---
	T72(40%MF)	20.2	3	-1%
	T72(60%MF)	26.0	14	27%
	T70(38%MF)	21.3	8	4%
425	T72(20%MF)	21.4	3	---
	T72(40%MF)	21.2	3	-1%
	T72(60%MF)	27.4	14	28%
	T70(38%MF)	22.0	9	3%
500	T72(20%MF)	21.2	3	---
	T72(40%MF)	21.1	3	-1%
	T72(60%MF)	26.7	14	26%
	T70(38%MF)	21.3	9	0%

Table 4.27 Mean Phase Angle across Stress State at 0.5 Hz and 20°C

Deviatoric Stress σ_D (kPa)	Mix Type	Mean Phase Angle δ (°)	Coefficient of Variation (%)	% Difference from T72(20%MF)
370	T72(20%MF)	17.9	2	---
	T72(40%MF)	18.4	3	3%
	T72(60%MF)	22.1	7	24%
	T70(38%MF)	19.0	5	6%
425	T72(20%MF)	19.9	1	---
	T72(40%MF)	20.1	3	1%
	T72(60%MF)	24.0	6	21%
	T70(38%MF)	20.7	5	4%
500	T72(20%MF)	19.9	1	---
	T72(40%MF)	20.2	2	2%
	T72(60%MF)	23.6	6	19%
	T70(38%MF)	20.2	4	1%

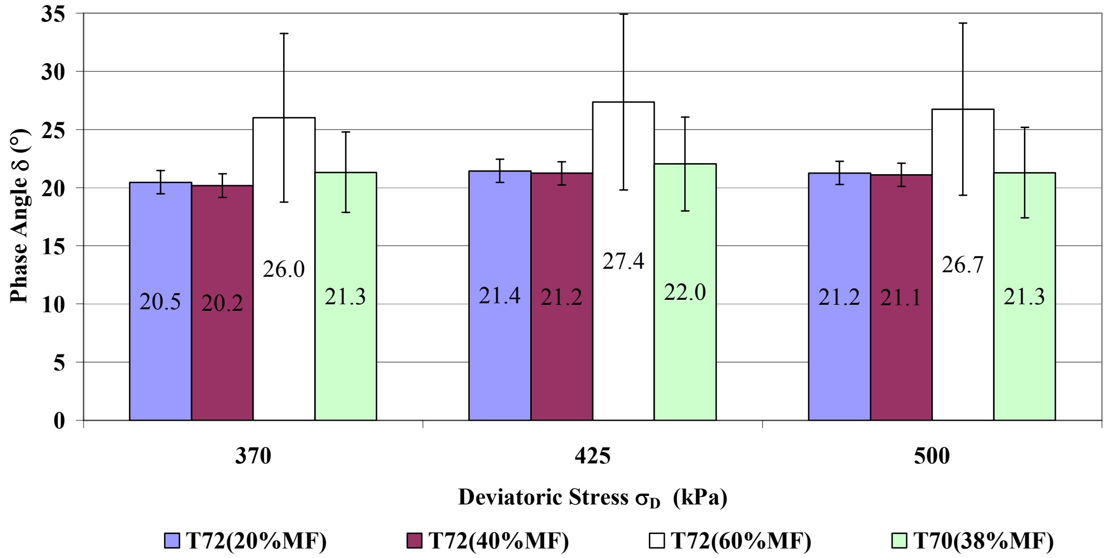

Figure 4.10 Mean Phase Angle across Stress State at 10 Hz and 20°C (± 2 SD)

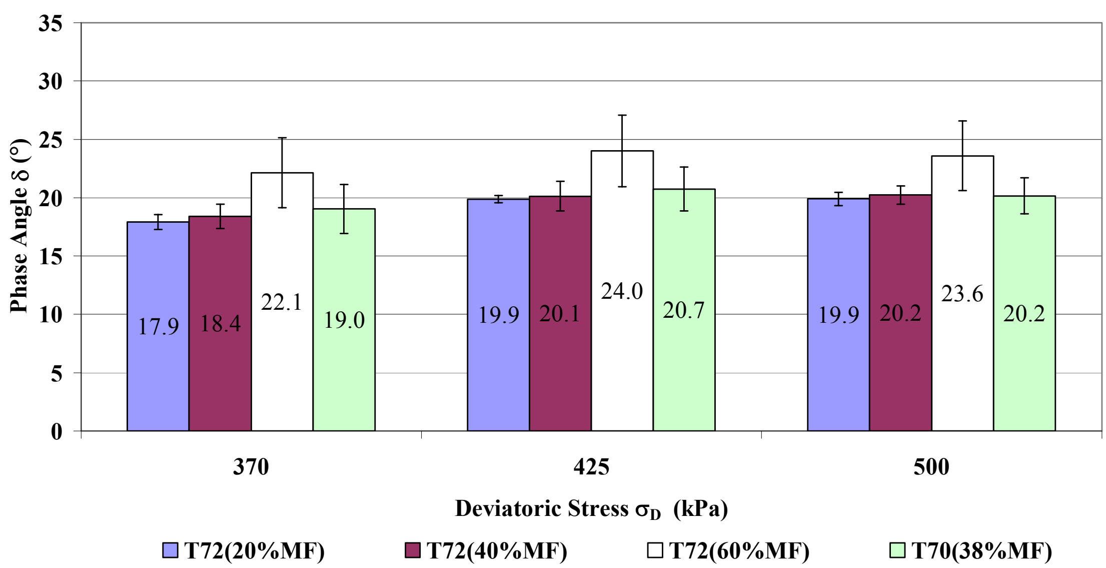

Figure 4.11 Mean Phase Angle across Stress State at 0.5 Hz and 20°C (± 2 SD)

Similar to the previous variables presented, the coefficient of variation for the Type 72 mix with 60 percent manufactured fines is 14 percent at the loading frequency of 10 Hz, which is higher than the CV for the other three mixes (see Table 4.26). At 0.5 Hz the variability in the phase angle across the ten repeat samples is similar for each of the mixes, with CV ranging between one and seven percent, as shown in Table 4.27.

The analysis of variance for phase angle at 10 Hz is shown in Table 4.28. Based on the ANOVA results, mix type significantly affects the phase angle at this frequency. Table 4.29 shows the ANOVA results of phase angle across mix type and deviatoric stress state at 0.5 Hz, indicating that both of these variables influence the phase angle at this frequency.

Table 4.28 Analysis of Variance for Phase Angle at 10 Hz and 20°C

Effect	Sum of Squares	Degrees of Freedom	Mean Squares	F-Test Statistic	P-value
Mix Type	*703.63*	*3*	*234.54*	*52.06*	*0.00*
Deviatoric Stress	21.33	2	10.66	2.37	0.099
Mix Type * Deviatoric Stress	3.54	6	0.59	0.13	0.99
Error	486.54	108	4.50		

Table 4.29 Analysis of Variance for Phase Angle at 0.5 Hz and 20°C

Effect	Sum of Squares	Degrees of Freedom	Mean Squares	F-Test Statistic	P-value
Mix Type	*306.70*	*3*	*102*	*117.44*	*0.00*
Deviatoric Stress	*78.56*	*2*	*39*	*45.12*	*0.00*
Mix Type * Deviatoric Stress	2.68	6	0.45	0.51	0.80
Error	94.01	108	0.87		

The results of Tukey's pairwise comparison for phase angle at 10 Hz are shown in Table 4.30. Table 4.31 shows the pairwise comparison results for phase angle at 0.5 Hz. At both frequencies, and at each stress state, the Type 72 mix with 60 percent manufactured fines has significantly higher phase angle than the other three mixes,

Table 4.30 Tukey's Homogeneous Groups for Phase Angle at 10 Hz and 20°C

Deviatoric Stress σ_D (kPa)	Mix Type	Mean Phase Angle δ (°)	Tukey's Homogeneous Groups	
			A	B
370	T72(40%MF)	20.2	****	
	T72(20%MF)	20.4	****	
	T70(38%MF)	21.3	****	
	T72(60%MF)	26.0		****
425	T72(40%MF)	21.2	****	
	T72(20%MF)	21.4	****	
	T70(38%MF)	22.0	****	
	T72(60%MF)	27.4		****
500	T72(40%MF)	21.1	****	
	T72(20%MF)	21.2	****	
	T70(38%MF)	21.3	****	
	T72(60%MF)	26.7		****

Table 4.31 Tukey's Homogeneous Groups for Phase Angle at 0.5 Hz and 20°C

Deviatoric Stress σ_D (kPa)	Mix Type	Mean Phase Angle δ (°)	Tukey's Homogeneous Groups				
			A	B	C	D	E
370	T72(20%MF)	17.9	****				
	T72(40%MF)	18.4	****				
	T70(38%MF)	19.0	****	****			
	T72(60%MF)	22.1				****	
425	T72(20%MF)	19.9		****	****		
	T72(40%MF)	20.1		****	****		
	T70(38%MF)	20.7			****	****	
	T72(60%MF)	24.0					****
500	T72(20%MF)	19.9		****	****		
	T70(38%MF)	20.2		****	****		
	T72(40%MF)	20.2		****	****		
	T72(60%MF)	23.6					****

ranging from 26.0 to 27.4 degrees across stress state at 10 Hz, and from 22.1 to 24.0 degrees across stress state at 0.5 Hz. There are no significant differences in phase angle between the Type 70 mix and the Type 72 mixes with 20 and 40 percent manufactured fines, respectively.

4.7 Significance of Results

This section presents an analysis of the parameters measured during the frequency sweep testing with respect to the reliability achieved with the number of repeat samples tested, by applying concepts of sample size analysis previously discussed in Section 3.3. In the absence of precedence, the acceptable margins of error have been set at approximately ten percent of the mean values of the mechanistic parameters evaluated. The assumed acceptable margins of error are listed in Table 4.32.

Table 4.32 Acceptable Margin of Error for Triaxial Frequency Sweep Properties

Property Measured	Acceptable Margin of Error
Dynamic Modulus, Ed (MPa)	200
Recoverable Axial Microstrain, RAMS, (10^{-6})	20
Recoverable Radial Microstrain, RMMS, (10^{-6})	10
Poisson's Ratio, ν	0.03
Phase Angle, δ (°)	2.0

4.7.1 Relationship of Level of Confidence to Sample Size

The results obtained from the frequency sweep testing can be used to estimate the relationship of sample size and level of confidence based on each variable measured, using the same formulations as were used for the volumetric and Marshall stability and flow testing in Section 3.3.2. The results of this estimation for dynamic modulus, recoverable axial microstrain, recoverable radial microstrain, Poisson's ratio, and phase angle, at 10 Hz and deviatoric stress of 500 kPa, at their respective acceptable margins of error, are shown in Figures 4.12 to 4.16, respectively. Tables showing the detailed results of the level of confidence analysis can be found in Appendix I.

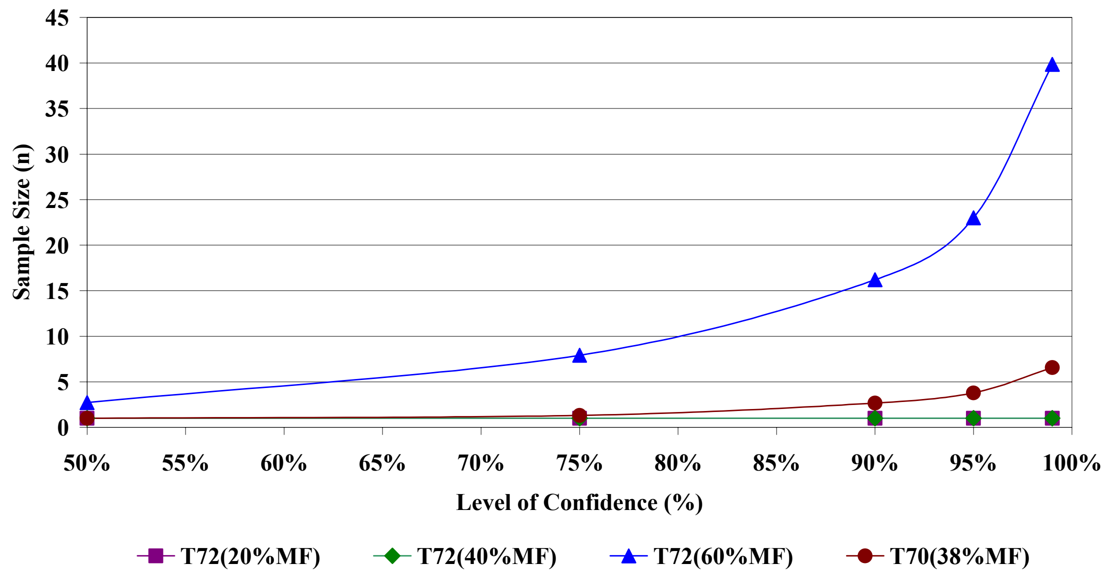

Figure 4.12 Relationship of Sample Size and Level of Confidence for Dynamic Modulus at 10 Hz and Deviatoric Stress of 500 kPa across Research Mixes at a Margin of Error of 200 MPa

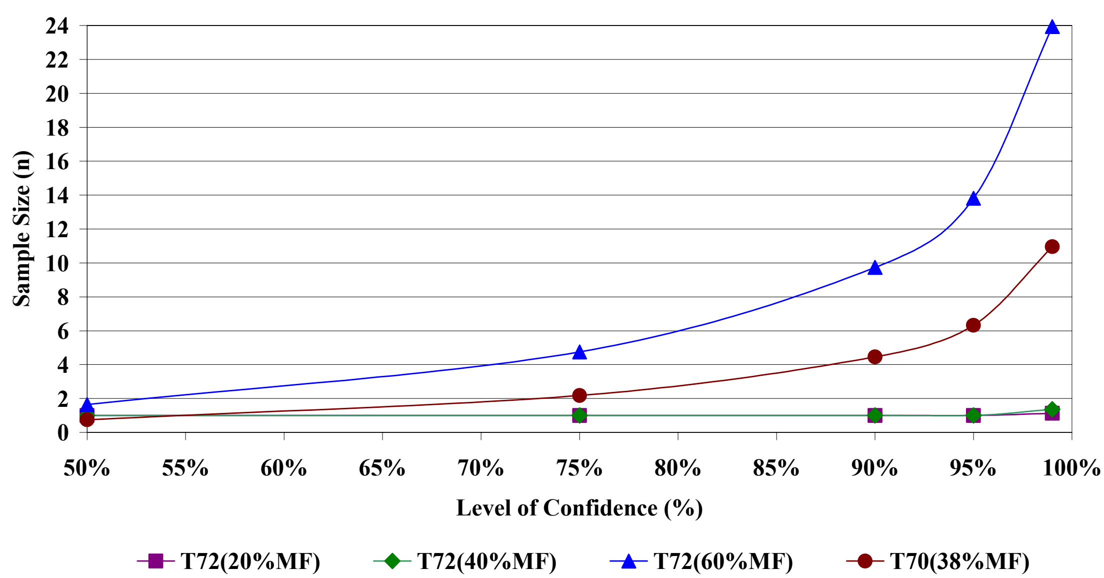

Figure 4.13 Relationship of Sample Size and Level of Confidence for Recoverable Axial Microstrain at 10 Hz and Deviatoric Stress of 500 kPa across Research Mixes at a Margin of Error of 20×10^{-6}

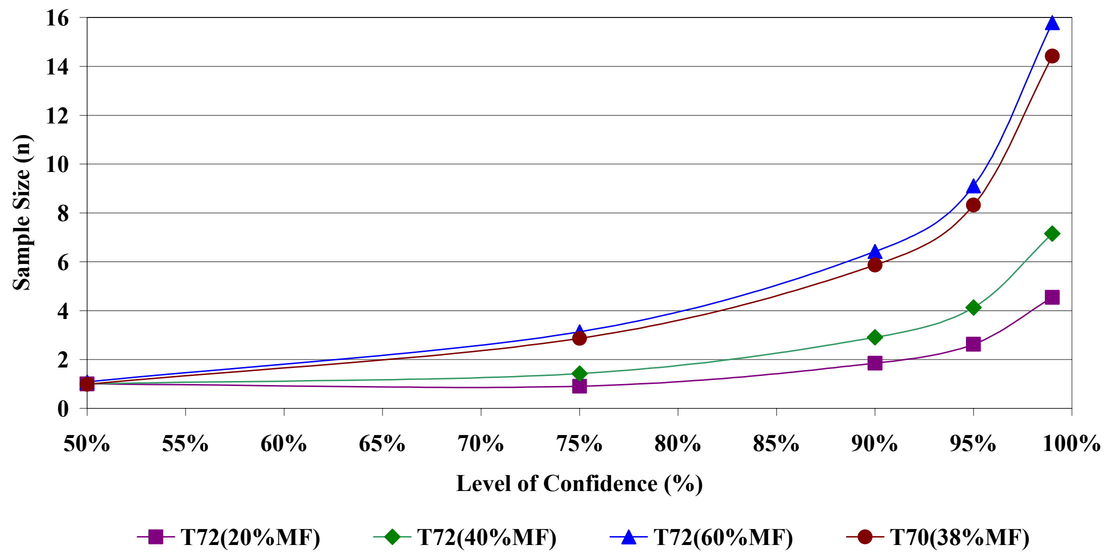

Figure 4.14 Relationship of Sample Size and Level of Confidence for Recoverable Radial Microstrain at 10 Hz and Deviatoric Stress of 500 kPa across Research Mixes at a Margin of Error of $10x10^{-6}$

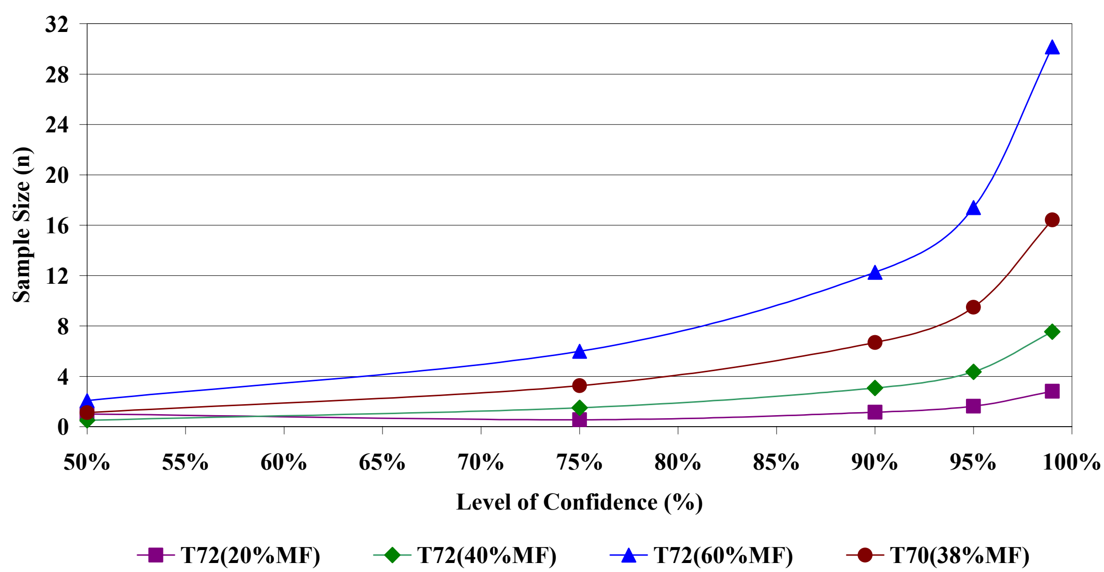

Figure 4.15 Relationship of Sample Size and Level of Confidence for Poisson's Ratio at 10 Hz and Deviatoric Stress of 500 kPa across Research Mixes at a Margin of Error of 0.03

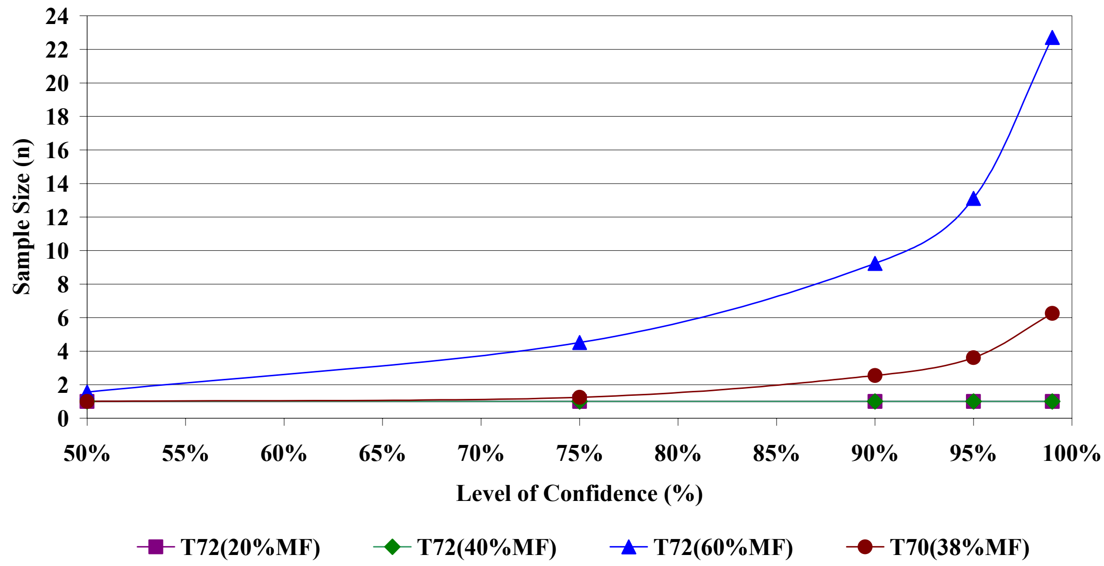

Figure 4.16 Relationship of Sample Size and Level of Confidence for Phase Angle at 10 Hz and Deviatoric Stress of 500 kPa across Research Mixes at a Margin of Error of 2 Degrees

Due to the variability in the triaxial frequency sweep results for Mix Type 72 with 60 percent manufactured fines, out of the four mix types, this mix requires the largest number of samples to achieve the desired level of confidence, while mix Type 72 with 20 percent manufactured fines requires the lowest number of repeat samples. Based on the curves shown in the above figures, assuming two repeat samples were to be tested, similar to standard SDHT laboratory practice for conventional mix characterization, the resulting level of confidence based on each the dynamic modulus, RAMS, Poisson's ratio, and phase angle for the four mixes would be a minimum of approximately 50 percent, respectively. Based on the RRMS, two repeat samples would yield an approximate level of confidence of 60 percent.

4.7.2 Level of Confidence Achieved

Based on the ten repeat samples tested in the RaTT cell for each mix type, a level of confidence achieved can be estimated based on the results of each mechanistic parameter, as previously discussed Section in 3.3.3.

Table 4.33 and Figure 4.17 illustrate the results of the level of confidence determination for each testing parameter, at the respective margin of error, at a frequency of 10 Hz, and a deviatoric stress of 500 kPa. As can be seen for the triaxial frequency sweep results, the lowest levels of confidence were achieved for mix type 72 with 60 percent manufactured fines, ranging from 80 to 96 percent, depending on the mechanistic parameter considered.

Table 4.33 Level of Confidence Achieved for Triaxial Frequency Sweep Properties across Research Mixes

Property Measured	Margin of Error	Estimated Level of Confidence (%)			
		T72(20%MF)	T72(40%MF)	T72(60%MF)	T70(38%MF)
Dynamic Modulus (MPa)	200	100.0	100.0	80.3	100.0
RAMS (10^{-6})	20	100.0	100.0	90.5	98.6
RRMS (10^{-6})	10	100.0	99.8	96.0	96.8
Poisson's Ratio	0.03	100.0	99.7	86.4	95.6
Phase Angle (°)	2.0	100.0	100.0	91.3	100.0

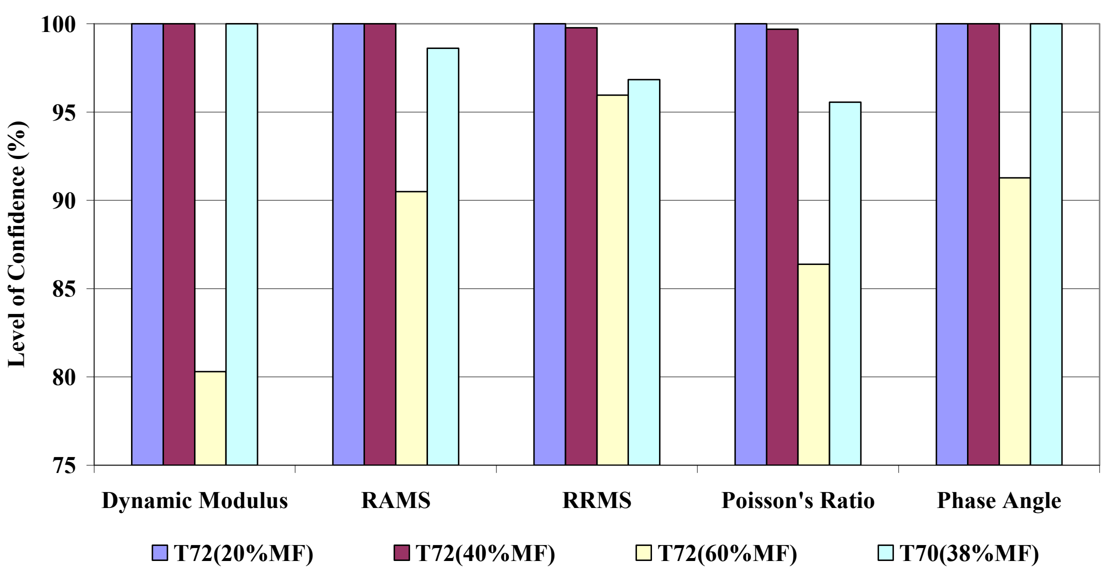

Figure 4.17 Level of Confidence Achieved for Triaxial Frequency Sweep Properties across Research Mixes

4.7.3 Experimental and Systematic Errors

As previously discussed, the level of experience and attention to detail by the laboratory personnel can significantly affect the number of repeat samples required as well as the accuracy and precision of test results. One of the benefits of the RaTT cell is the fact that it is fully computer-controlled, and once the sample is placed in the testing apparatus, human interaction is eliminated. Although the testing equipment and settings were inspected by the author prior to testing, some of the systematic errors that may have affected the sample response to loading and frequency include:

- Variability in the confining pressure applied to the sample.
- Improper placement of LVDTs.
- Calibration and/or feedback control variability in the RaTT cell.

4.8 Chapter Summary

This chapter presented the results of triaxial frequency sweep analysis conducted using the RaTT cell. The testing was performed at 20°C, at loading frequencies ranging from 10 Hz to 0.5 Hz, and at deviatoric stress states of 370 kPa, 425 kPa, and 500 kPa. Dynamic modulus, recoverable axial microstrains, recoverable radial microstrains, Poisson's ratio, and phase angle were determined during testing, and several interesting trends were observed.

Increasing the manufactured fines content from 20 to 40 percent of total fines in the Type 72 mix did not have significant effects on dynamic modulus. Increasing the manufactured fines content of the Type 72 mix from 20 to 60 percent of total fines resulted in a significant increase in the dynamic modulus, across the three stress states, at both frequencies of axial loading (51 to 52 percent increase at 10 Hz, and 24 to 27 percent at 0.5 Hz). The dynamic modulus for the Type 70 mix did not differ from the Type 72 mixes with 20 and 40 percent manufactured fines regardless of stress state and/or frequency. Dynamic modulus reduced in magnitude by approximately 50 percent when the testing frequency was reduced from 10 Hz to 0.5 Hz. In addition, as deviatoric stress increased, dynamic modulus decreased.

Increasing the manufactured fines content from 20 to 40 percent of total fines in the Type 72 mix did not have significant effects on recoverable axial microstrains. Increasing manufactured fines content of the Type 72 mix from 20 to 60 percent of total fines resulted in significantly lower recoverable axial microstrains, across the three stress states, at both frequencies of axial loading (32 to 33 percent reduction at 10 Hz, and 18 to 20 percent at 0.5 Hz). It is evident at both frequencies and all stress states that mix type 70 did not have statistically different RAMS than the Type 72 mixes with 20 and 40 percent manufactured fines, respectively. As frequency was lowered to 0.5 Hz, RAMS increased in magnitude by approximately 50 percent. Also, an increase in deviatoric stress resulted in increased recoverable axial microstrains.

Increasing the manufactured fines content from 20 to 40 percent of total fines in the Type 72 mix did not have significant effects on recoverable radial microstrains. Increasing manufactured fines content of the Type 72 mix from 20 to 60 percent of total fines resulted in significantly lower recoverable radial microstrains, across the three stress states, at the loading frequency of 10 Hz (18 to 22 percent reduction). At the axial loading frequency of 0.5 Hz, the benefits of increased manufactured fines are only significant when the deviatoric stress is 500 kPa, where the Type 72 mix with 60 percent manufactured fines results in reduced RRMS (reduction of 14 percent) when compared to the Type 72 mix with 20 and 40 percent manufactured fines, respectively. The RRMS for the Type 70 mix are statistically the same as the RRMS for the Type 72 mix with 20 and 40 percent manufactured fines, respectively, at each stress state and frequency. As frequency was lowered to 0.5 Hz, radial microstrains increased in magnitude by approximately 100 percent. Also, as deviatoric stress increased, RRMS increased.

Increasing the manufactured fines content from 20 to 40 percent of total fines in the Type 72 mix did not have significant effects on Poisson's ratio. Increasing manufactured fines content of the Type 72 mix from 20 to 60 percent of total fines resulted in significantly higher Poisson's ratio at 10 Hz (20 to 23 percent increase), at deviatoric stress states of 425 and 500 kPa, respectively. The increase in manufactured fines content in the Type 72 mix from 20 to 60 percent of total fines resulted in increased Poisson's ratio (13 percent increase) at the axial loading frequency of 0.5 Hz, and deviatoric stress state of 370 kPa. As the deviatoric stress was increased to 425 and 500 kPa, with the combined effects of the slower loading, the influence of mix type was no longer significant. As frequency was lowered to 0.5 Hz, Poisson's ratio increased in magnitude by approximately 25 percent, with mean values from 0.43 to 0.49, depending on mix type and deviatoric stress state.

Increasing the manufactured fines content from 20 to 40 percent of total fines in the Type 72 mix did not have significant effects on phase angle. Increasing manufactured fines content of the Type 72 mix from 20 to 60 percent of total fines resulted in significantly higher phase angle across the three stress states, at both frequencies of axial loading (26 to 28 percent increase at 10 Hz, and 19 to 24 percent at 0.5 Hz). It is evident at both frequencies and all stress states that mix type 70 did not have statistically different phase angle than the Type 72 mixes with 20 and 40 percent manufactured fines, respectively. Also, phase angle increased as deviatoric stress was increased, particularly at the axial loading frequency of 0.5 Hz.

The variability within the Type 72 mix with 60 percent manufactured fines was significantly higher than within the other three mixes tested for most of the parameters measured, as was indicated by high coefficients of variation within the ten repeat samples (15 to 18 percent for dynamic modulus, 12 to 18 percent for RAMS, 11 to 20 percent for RRMS, 10 to 18 percent for Poisson's ratio, and 6 to 14 percent for phase angle). This is further evident by the lower levels of confidence achieved in frequency sweep results for the Type 72 mix with 60 percent manufactured fines (from 80 to 91 percent), when compared to the other mixes. This behaviour could be caused by the increased amount of fractured aggregate, which increases the importance of particle arrangement within the sample, therefore increasing the variability between the samples.

CHAPTER 5 ECONOMIC IMPLICATIONS OF 12.5 MM TOP SIZE MIXES WITH INCREASED MANUFACTURED FINES CONTENT

Along with quantifying the engineering improvements of selecting smaller top size and higher manufactured fine aggregate content in Saskatchewan HMAC mixes, there is a need to explicitly quantify the economic benefits of implementing such changes. This chapter contains a brief examination of the potential costs and benefits of selecting well-performing, 12.5 mm top size mixes on the life cycle costs of SDHT paved roads. The impacts on provincial aggregate management and on the provincial economy are also discussed.

5.1 Preservation of Road Assets

As previously explained in Chapter Two, the SDHT annual provincial paving budget consists of approximately $44 million spent on placing approximately 600,000 tonnes of asphalt concrete. Improving the performance of pavements and therefore reducing their life cycle costs can result in significant savings in funding required to maintain an acceptable level of service. It is assumed that implementing finer mixes, such as the Type 72 mix, with increased manufactured fines, results in improved rut-resistance of SDHT asphalt concrete pavements. Life cycle cost analysis was performed to quantify the potential savings in preservation costs for a 25 year pavement life based on these assumptions.

The impacts on preservation costs were assessed based on three pavement performance scenarios. These scenarios are: failed pavement, typical SDHT pavement, and well-performing pavement. An assumption was made that the initial construction costs and the annual routine maintenance costs remain the same regardless of performance. In all three cases the road has one travel lane per direction. It was assumed that in all three scenarios the structural design of the pavement was adequate for the existing field state conditions. Treatments were selected based on current asset management practice, and treatment costs were set in consultation with SDHT asset management and construction staff. Details of the analysis are shown in Appendix J. Figures 5.1 to 5.3 illustrate the preservation cash flow that would be required for each performance scenario.

Figure 5.1 shows the preservation costs for a failed pavement. The pavement is assumed to have failed in rutting in the first five years of service life. This scenario is based on previous SDHT pavement mixes that resulted in plastic flow rutting. For the purposes of this analysis, it is estimated that five percent of SDHT pavement network will experience problems with premature rutting under current practices of design, construction and preservation.

Figure 5.2 shows the preservation costs for a typical SDHT pavement. This pavement is assumed to result in poor rutting after fifteen years of service, which is based on the current target service life used by SDHT for structural design. The majority of Saskatchewan pavements would follow this trend if no work is done to mitigate the rutting conditions at an earlier stage. For the purposes of this analysis, it is estimated that 85 percent of SDHT pavement network performs in this manner if current practices are maintained.

Figure 5.3 shows the life cycle cost scenario for a well-performing pavement. This pavement is engineered well enough to remain in good rutting condition over the 25 year life cycle. This scenario assumes that the asphalt mix has been engineered to be rut-resistant, for example by selecting a dense graded asphalt mix with 12.5 mm top size and a large amount of manufactured materials. For the purposes of this analysis, it is estimated that ten percent of SDHT pavement network performs in this manner under status quo conditions.

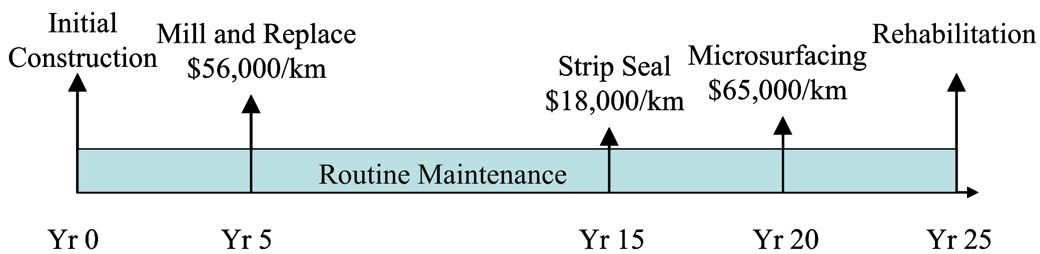

Figure 5.1 Preservation Costs for a Failed Pavement - Plastic Flow Rutting in the First 5 Years of Service Life

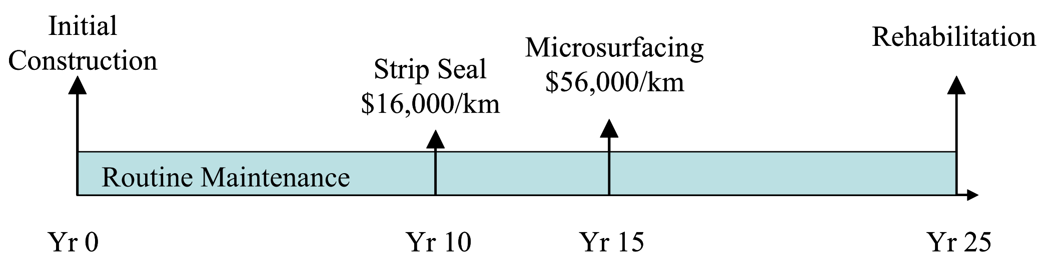

Figure 5.2 Preservation Costs for a Typical SDHT Pavement – Poor Rutting in Year 15 of Service Life

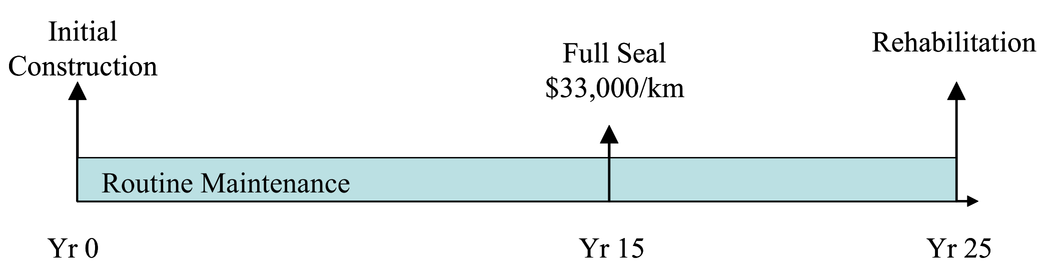

Figure 5.3 Preservation Costs for a Well-Performing Pavement – Rutting Remains Good over 25 Years of Service Life

The treatment costs shown in the cash flow diagrams can be determined in present value dollars, by applying the following present value calculation to each future treatment cost:

$$PV = \frac{C}{(1+i)^t} \tag{5.1}$$

where:

PV = Present Value (dollars)

C = Future Amount (dollars)

t = Number of Terms (years)

i = Interest Rate (percent)

The annual treatment costs for each pavement performance scenario can be discounted to today's dollars using the above equation, and added together to determine the total preservation costs. Table 5.1 illustrates a summary of the life cycle costs associated with routine maintenance and capital preservation, not including the initial construction cost and the rehabilitation cost at the end of the 25-year life cycle, for each of the pavement performance scenarios. Based on direction from SDHT asset management staff, an interest rate of 3 percent was applied, and inflation was not considered. An asphalt concrete pavement with premature rutting failure in the first five years of service life results in an increase in life cycle costs of approximately \$75,000 per kilometre when compared to a well-performing pavement, as can be seen in Table 5.1. Improving asphalt concrete rutting performance of typical SDHT pavements could result in an approximate reduction of \$26,000 (or 96 percent), in life cycle costs per kilometre of road.

The potential cost savings of improving pavement performance can be quantified by examining the possible changes to the resulting pavement quality from annual network rehabilitation. SDHT plans to rehabilitate 490 km of the provincial pavement network during the 2007-08 construction season (Kwon 2007). Assuming that SDHT constructs 500 km of asphalt concrete paved roads annually, two levels of performance of these annually constructed HMAC pavements are evaluated: the status quo level, and the improved level. Table 5.2 shows a summary of the life cycle preservation costs for the two levels of pavement performance.

Table 5.1 Summary of Preservation Costs over Pavement Life Cycle in Present Value Dollars Per Kilometre of Road

	Routine Maintenance Costs	Capital Preservation Costs	Total Preservation Costs	Difference from Well-Performing Pavement	% Increase from Well-Performing Pavement
Failed Pavement	$6,068	$96,117	$102,184	$74,757	+273%
Typical Pavement	$6,068	$47,573	$53,641	$26,214	+96%
Well-Performing Pavement	$6,068	$21,359	$27,427	---	---

Table 5.2 25 Year Pavement Life Cycle Preservation Costs at Different Performance Levels for 500 km of HMAC Roads Paved Annually by SDHT

	Life Cycle Preservation Costs ($/road km)	Status Quo Performance		Improved Performance	
		Road Length in Pavement Category (km)	Life Cycle Costs (PV $)	Road Length in Pavement Category (km)	Life Cycle Costs (PV $)
Failed Pavement	$102,184	25	$2,554,600	0	$0
Typical Pavement	$53,641	425	$22,797,425	212	$11,371,892
Well-Performing Pavement	$27,427	50	$1,371,350	288	$7,898,976
Total	**---**	**500**	**$26,723,375**	**500**	**$19,270,868**

PV – Present Value

The status quo level of performance would result in five percent of failed pavements (life cycle costs as shown in Figure 5.1), 85 percent of typical performing pavements (Figure 5.2), and ten percent of well-performing pavements (Figure 5.3).

Under the improved level of performance, the analysis assumes that with rut-resistant HMAC mixes, SDHT can eliminate the premature pavement rutting failures and shift this portion of SDHT roads into the well-performing category, and that 50 percent of the typical SDHT pavements, which result in poor rutting after 15 years, can also be improved to the level of well-performing pavements. This would result in 42.5 percent of typical SDHT performing pavements, and 57.5 percent of roads in the well-performing life cycle cost scenario.

By improving the rutting performance of 500 km of roads annually from the status quo to the improved performance level, SDHT can reduce the long term pavement life cycle preservation costs of these roads by approximately $7.5 million on an annual basis. A rate of 500 km of re-paved asphalt concrete roads per year results in an 18 year rehabilitation cycle for the 8,975 km of the provincial pavement network. Assuming SDHT continues to place approximately 500 km of new HMAC surface for the next 18 years, this amount translates to an astounding $102.5 million in present value dollars, as shown in Table 5.3.

In order to evaluate the savings in preservation spending that can be realized by engineering rut-resistant mixes, the routine maintenance costs for all three pavement performance scenarios were assumed to remain the same, regardless of changes in performance and varying application of capital preservation treatments. However, this assumption is conservative, because it follows that if the asphalt concrete mix is engineered well, then routine maintenance will be reduced. It is not unreasonable to expect that the routine maintenance costs would decrease by ten percent for the well-performing pavements. Also, the above analysis does not address reduced user costs due to improved road conditions, such as minimized time delays due to road repairs, and decrease in vehicle maintenance costs. As can be seen, even based on these conservative assumptions, improving HMAC mix performance has the potential to result in substantial preservation cost savings to the Province.

Table 5.3 Potential Savings in Pavement Life Cycle Preservation Cost through Improving the Rutting Performance of HMAC Roads Paved Annually by SDHT

	Potential Cost Savings in Present Value Dollars
Annual Savings	$7,452,507
Savings After 18 years	$102,498,152

5.2 Impacts on Aggregate Resource Management

As previously discussed in Chapter Two, all of Saskatchewan's HMAC aggregates are manufactured from surficial glacial gravel sources. Existing quality gravel pits suitable for HMAC aggregate production are being exhausted, and it is becoming increasingly difficult to locate new aggregate sources. In fact, an SDHT aggregate management strategy review estimated that $193.3 million cubic metres of quality aggregate will be required to meet the provincial needs up to the year 2049, while it is estimated that the Province currently has access to 150 million cubic metres in available gravel sources of varying quality (SDHT 2001-A). Therefore, there is a potential shortage of aggregate supply of approximately 43.3 million cubic metres (73.6 million metric tonnes) to meet the needs of the next 42 years.

In addition to potential savings due to improved pavement performance and extended performance life cycle, manufacturing mixes with smaller top size should result in better source utilization, and increased amount of manufactured material available for HMAC production.

5.2.1 Gravel Source Utilization

A typical aggregate manufacturing process involves screening off natural material smaller than 9 mm prior to the crushing stage. It is also common practice to remove "pea gravel" (ranging in size from 9 mm up to the top size of the mix being produced), since it is too small to be fractured in the crushing process. The remaining larger rocks are crushed, and the resulting manufactured material is typically split on the 5 mm sieve, into a manufactured fines and a manufactured coarse pile. If the pea-sized aggregates are not removed, the manufactured coarse material is not likely to meet the high coarse fracture requirements specified by SDHT (as high as 95 percent, depending on aggregate type and mix design type). The rejected pea-sized rocks are stockpiled in the gravel pit as waste.

Theoretical crushing analysis of four randomly selected SDHT gravel pits was completed to examine the changes in aggregate quantities resulting from manufacturing the two different hot mix aggregate structures examined in this research project (Halldorson 2007). This type of analysis is routinely used by SDHT to assess the gravel sources for suitability towards manufacturing various types of aggregate required for highway maintenance and rehabilitation. Table 5.4 provides a summary of the resulting amounts of manufactured fine and coarse aggregate, as well as the amount of pea sized aggregate that would typically have to be rejected to meet minimum requirements of coarse fracture for the production of the Type 70 mix and for the Type 72 mix.

Table 5.4 Theoretical Aggregate Crushing Analysis for Selected SDHT Gravel Sources

	Percent Volume of Parent Pit Run Aggregate			
Gravel Source	**62K-097**	**73C-132**	**72P-178**	**72O-051**
Type 70 Aggregate				
Manufactured Coarse Aggregate	9%	18%	14%	17%
Manufactured Fine Aggregate	7%	14%	11%	13%
Pea gravel retained on 9 mm sieve	9%	12%	20%	14%
Total Manufactured Material	**16%**	**33%**	**26%**	**30%**
Type 72 Aggregate				
Manufactured Coarse Aggregate	8%	18%	15%	16%
Manufactured Fine Aggregate	11%	22%	19%	20%
Pea gravel retained on 9 mm sieve	5%	5%	12%	7%
Total Manufactured Material	**19%**	**40%**	**34%**	**37%**

Based on the parent gradations of the gravel sources examined, theoretical analysis showed that the amount of rejected pea-sized material can be reduced by as much as 58 percent when manufacturing the 12.5 mm top size Type 72 aggregate instead of the 18 mm top size Type 70 aggregate (Figure 5.4). Further, since the salvaged pea gravel is processed, manufacturing Type 72 aggregate results in a three to eight percent increase in total manufactured aggregate produced in each gravel source, as is illustrated in Figure 5.5.

Considering that SDHT used approximately 600,000 metric tonnes of asphalt concrete during the 2005-06 paving season, an increase of three percent of useful material obtained from the crushing process translates into an annual savings of 18,000 metric tonnes, and an increase of eight percent saves 48,000 metric tonnes, annually. If the period from 2007 to 2049 is considered, which coincides with the time period used in the SDHT aggregate needs study previously mentioned (SDHT 2001-A), these savings amount to 2,016,000 metric tonnes of aggregate, as shown in Table 5.5.

The increase in the amount of manufactured aggregate resulting from crushing to a smaller top size can be translated into monetary savings. Assuming an aggregate manufacturing cost of $15 per metric tonne, this additional aggregate is worth $270,000 and $720,000, respectively, on an annual basis. The potential value of the aggregate saved over the next 42 years is $1.7 million in present value dollars.

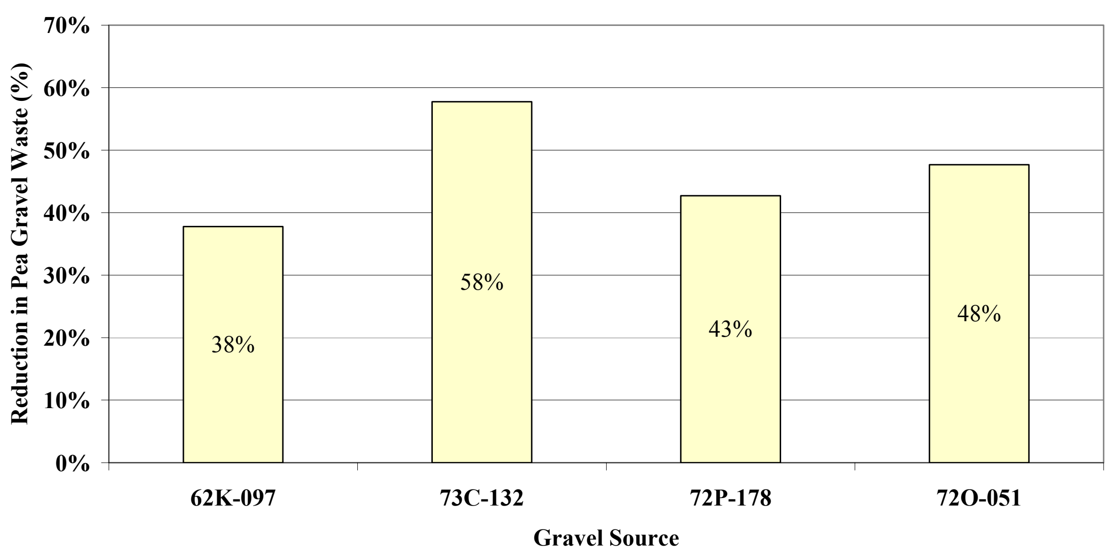

Figure 5.4 Percent Reduction in Pea Gravel Waste in Selecting Type 72 Aggregate compared to Type 70 Aggregate

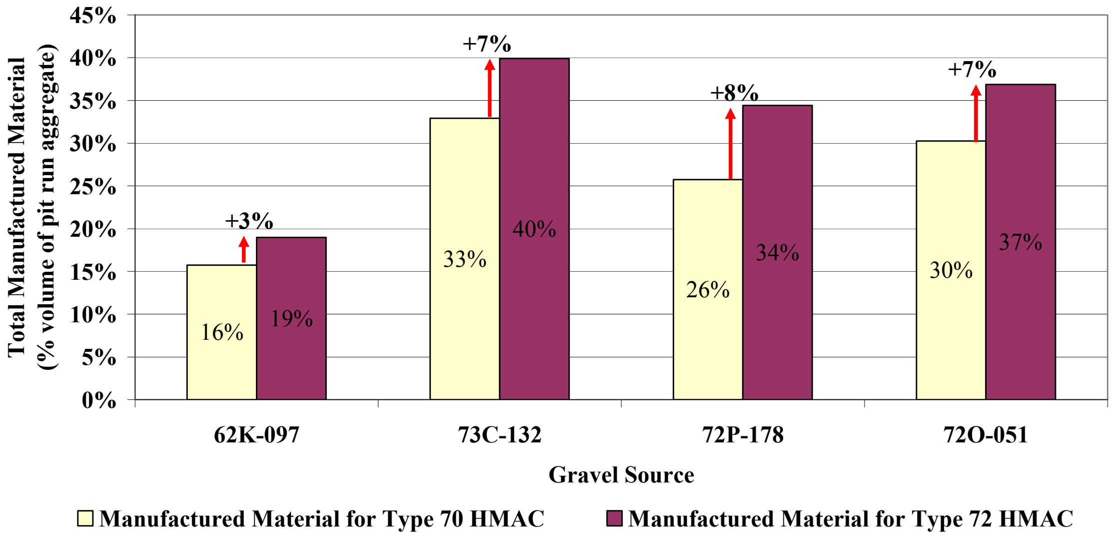

Figure 5.5 Percent Gravel Source Utilization for Type 70 and Type 72 Aggregate

This estimate is conservative, because it assumes the sources are owned by the Province, and does not take into account the cost of purchasing the aggregate from private owners. Although most sources currently used by SDHT are owned or leased by the Province, there is a possibility that aggregate may need to be purchased from private sources. Also, the value of this additional aggregate in the context of pit depletion is not included. There are also potential monetary savings associated in maximizing aggregate utilization in terms of its future availability for other projects.

It is difficult to determine the difference in costs for crushing Type 70 aggregate when compared to Type 72 aggregate based on historic crushing costs, because the bid items vary between contracts, and some costs include haul and/or mobilization. The possibility of increased crushing costs to manufacture smaller top size aggregate with higher fracture was discussed with Saskatchewan contractors and SDHT construction staff. Based on those discussions, it is reasonable to assume that the additional crushing costs, if any, would be in the order of $1 per metric tonne of aggregate. Based on the 600,000 metric tonnes of aggregate used by SHDT in the 2005-06 construction season, this increase would amount to $600,000 annually. However, this additional cost would likely be offset by the direct savings realized from maximizing pit utilization, which as previously discussed and shown in Table 5.5, could range from $270,000 to $720,000.

Table 5.5 Potential Savings in Gravel Source Utilization from Selecting Type 72 Aggregate Instead of Type 70 Aggregate

% Increase in Useful Aggregate from Source	Annual Savings of Aggregate (Metric Tonnes)	Savings over a 42 Year Period from 2007 to 2049 (Metric Tonnes)
Minimum 3%	18,000	756,000
Maximum 8%	48,000	2,016,000

Further examination of the theoretical crushing analysis and discussions with SDHT laboratory staff suggest that crushing for a smaller top size results in a larger ratio of manufactured fine to manufactured coarse aggregate (Bray 2006). For example, typical crushing operations for Type 70 aggregate result in approximately 0.6:1 ratio of fine to coarse manufactured aggregate (manufacturing 100 tonnes of coarse aggregate results in 60 tonnes of manufactured fine aggregate). Manufacturing Type 72 aggregate can result in as high a ratio as 1:1 of manufactured fines to manufactured coarse aggregate.

Field performance has shown the value of manufactured materials in hot mix asphalt. As a result, the mix design process attempts to fully utilize the manufactured aggregate available, and to maintain the same ratio of manufactured fine to manufactured coarse aggregate in the mix design as was achieved during the crushing process. This approach minimizes aggregate waste, and ensures that the mechanically fractured materials will be used up as best as possible during HMAC manufacturing. The fact that crushing to 12.5 mm top size results in a higher ratio of manufactured fine to coarse aggregate would therefore result in increased amounts of total manufactured aggregate and reduced amounts of natural sands in the Type 72 mix design gradation.

5.2.2 Reduced Aggregate Needs for Pavement Preservation

Important aggregate savings can be realized if the life cycle performance of SDHT asphalt concrete pavements can be improved. Roads that are performing well are less susceptible to damage, and therefore require fewer preservation treatments in their 25 year life cycle. This in turn results in a reduction of aggregate needs for preservation treatments. Approximately 1,000 metric tonnes of aggregate per kilometre of road can be saved by eliminating the need for the removal and replacement of an asphalt layer that is failing in plastic flow rutting. Also, based on the three pavement life cycle scenarios considered (Figure 5.1to Figure 5.3), improving typical pavement performance to the well-performing pavement category level eliminates the need for strip seals, which requires approximately 114 metric tonnes of aggregate per kilometre of road.

The financial implications of reduced aggregate requirements for preservation treatments can be assessed by once again examining the 500 km of roads that SDHT surfaces with hot mix asphalt concrete on an annual basis. Table 5.6 shows the volume of aggregate required to maintain the annually paved 500 km of road over the 25 year life cycle, based on the current status quo level of performance, and based on the improved level, previously introduced in Section 5.1. The resulting 25 year life cycle aggregate requirements for the 500 km of newly paved road decrease by 29 percent when pavement performance is improved (decrease from 183,390 metric tonnes, to 129,618 metric tonnes). Therefore, every year that SDHT can create 500 km of well-performing pavements; an aggregate saving of approximately 53,772 metric tonnes is realized.

Table 5.6 25 Year Life Cycle Aggregate Requirements at Different Performance Levels for 500 km of HMAC Roads Paved Annually by SDHT

	Status Quo HMAC Performance		Improved HMAC Performance	
	Portion of Annually Paved Roads (Km)	**Aggregate Required over Life Cycle (Metric Tonnes)**	**Portion of Annually Paved Roads (Km)**	**Aggregate Required over Life Cycle (Metric Tonnes)**
Failed Pavement	25	34,763	0	0
Typical Pavement	425	138,083	212	68,879
Well-Performing Pavement	50	10,545	288	60,739
Total	**500**	**183,390**	**500**	**129,618**

Table 5.7 further examines the potential aggregate savings resulting from the reduced needs for preservation treatments, along with their monetary value. The annual reduction in aggregate needs for maintenance treatments translates to a potential saving of 2.26 million metric tonnes of aggregate over the period between 2007 and 2049, the monetary value of which is estimated at $19.1 million present day dollars.

Table 5.7 Potential Aggregate Savings from Reducing the Need for Preservation Treatments During the Life Cycle of HMAC Roads Paved Annually by SDHT

Time Frame	Potential Aggregate Savings	
	Volume of Aggregate Saved (Metric Tonnes)	Value of Aggregate (PV $)
Annual	53,772	$806,580
After 18 years	967,896	$11,093,309
Between 2007 and 2049	**2,258,424**	**$19,117,042**

PV – Present Value

5.3 Other Impacts on the Provincial Economy

Along with the direct cost savings that can potentially be realized by SDHT through implementing well-performing, rut resistant, asphalt concrete mixes by selecting smaller top size and higher manufactured fines content aggregates, multiple other benefits that are of importance need to be acknowledged.

Saskatchewan economy is highly dependent on road transportation. The Province exported and imported approximately $50 billion of goods and services in 2004, the majority of which was moved by road. Ensuring the efficient and safe movement of goods is an important issue for Saskatchewan. Further, the average Saskatchewan resident relies primarily on road transportation for daily activities, and would therefore also benefit from improved road performance (Roadbuilders Saskatchewan 2005).

The implications of improved performance of asphalt concrete surfaced pavements are far-reaching. Better roads translate into decreased vehicle maintenance costs for the road users, and time savings due to the reduced amount of road preservation work necessary to maintain an acceptable level of service. Improved road surface conditions result in lower fuel consumption, and therefore in a reduction of the impacts of transportation of goods and people on the environment.

5.4 Chapter Summary

This chapter briefly discussed the potential economic implications of implementing mixes with smaller top size and increased amounts of manufactured material. Specifically, the potential for reduced life cycle costs due to improved performance of the newly placed asphalt concrete mixes was examined, based on the assumption that SDHT surfaces approximately 500 km of road annually with asphalt concrete. Also, the implications of crushing to a smaller top size were evaluated by analysing the impacts of crushing for Type 70 aggregate and for Type 72 aggregate on four randomly selected SDHT gravel sources. The decline in demand for aggregate due to reduced need for preservation treatments was also quantified by examining the effects of improved performance on the 500 km of road annually paved by SDHT. Finally, the implications of engineering well-performing asphalt concrete mixes on the provincial economy were briefly discussed.

It was found that improving the rutting performance of asphalt concrete mixes in Saskatchewan could result in approximately 96 percent reduction in annual preservation costs for the majority of Saskatchewan asphalt concrete surfaced roads. It was also determined that asphalt concrete mixes which result in plastic flow rutting, and therefore premature failure, cost approximately 273 percent more to maintain over a 25 year life cycle than well-performing pavements.

By examining the aggregate manufacturing process and comparing the implications of using Type 72 aggregate versus Type 70 aggregate, it was determined that significant aggregate savings can result from crushing to the 12.5 mm top size of aggregate for the Type 72 mix, instead of to the 18 mm top size for Type 70 HMAC aggregate. Specifically, there is a potential of reducing the amount of rejected pea gravel by 38 to 58 percent, depending on the gravel source. Also, since the salvaged pea gravel is put through the crushing process, crushing to the 12.5 mm top size results in an increase of total manufactured material produced, in the order of three to eight percent of the volume of parent pit run aggregate. There is a potential to save over 2 million metric tonnes of aggregate over a period of the next 42 years by selecting the smaller top size aggregate. Further, the ratio of manufactured fine aggregate to manufactured coarse aggregate increases when aggregate is crushed to the 12.5 mm top size, resulting in better opportunity to maximize the benefits of the manufactured material in the asphalt concrete aggregate skeleton.

The improved life cycle performance of rut-resistant asphalt concrete mixes also results in a reduction of aggregate required for preservation treatments during the pavement life cycle. Close to 11 million metric tonnes of HMAC aggregate can be saved during a period of 18 years by reducing demands for preservation treatments, assuming the pavement performance level shifts from status quo to the improved level.

A summary of the potential cost savings resulting from implementing rut resistant, well-performing mixes with 12.5 mm top aggregate size is presented in Table 5.8. Potential cost savings after 18 years of paving 500 km per year with rut-resistant, well-performing HMAC mixes amount to $112.4 million in present value dollars. There is a potential to save approximately $193.7 million in the next 42 years.

Table 5.8 Summary of Potential Cost Savings Resulting From Implementing Well-Performing Mixes with 12.5 mm Top Size Aggregate

	Annual Savings ($)	Savings over 18 Years (PV $)	Savings from 2007 to 2049 (PV $)
Reduced Life Cycle Costs	$7,452,507	$102,498,152	$176,634,545
Improved Gravel Source Use	$720,000	$9,902,529	$17,064,979
	$8,172,507	**$112,400,681**	**$193,699,524**

PV – Present Value

Table 5.9 shows the potential savings in aggregate quantity that can result from implementing well-performing 12.5 mm top size mixes. As can be seen, there is an opportunity to save approximately 4.3 million metric tonnes of aggregate in the next 42 years. Based on estimates that the currently available aggregate sources will fall approximately 73.6 million metric tonnes short of the volume of aggregate required to meet the provincial needs up to the year 2049, the total potential savings of 4.3 million metric tonnes could compensate for approximately six percent of the short fall in provincial aggregate needs for the period between 2007 and the year 2049.

Table 5.9 Summary of Potential Savings in Aggregate Volume if Well-Performing Type 72 HMAC Mixes are implemented by SDHT

Source of Savings	Annual Aggregate Savings (Metric Tonnes)	Aggregate Savings From 2007 to 2049 (Metric Tonnes)
Crushing to 12.5 mm Top Size	48,000	2,016,000
Reduced Need for Preservation Treatments	53,772	2,258,424
	101,772	**4,274,424**

CHAPTER 6 CONCLUSIONS AND RECOMMENDATIONS

Saskatchewan Highways and Transportation relies on dense-graded hot mix asphalt concrete mixes for construction and rehabilitation of asphalt pavement surfaced highways. As a result of increased commercial truck traffic on the provincial road network, over the last two decades, some of Saskatchewan's recently placed dense graded HMAC pavements have been observed to show a susceptibility to premature permanent deformation in the asphalt mix. One of the aggregate properties thought to have significant influence on mix performance under traffic loading is the shape of the aggregate. Specifically, the physical properties of the fine aggregate (smaller than 5 mm in diameter) are of particular importance in dense graded mixes. Although empirical evidence suggests that there are performance benefits associated with using angular fine aggregate, the relationship of this parameter on mechanistic mix performance and resistance to permanent deformation has not yet been clearly defined.

The primary objective of this research has been to conduct laboratory analysis to determine the physical, empirical, and mechanistic behaviour sensitivity to the proportion of manufactured and natural fine aggregate in SDHT Type 72 hot mix asphalt concrete. The second objective of this research has been to compare the mechanistic behaviour of the Type 72 mixes considered in this research to conventional SDHT Type 70 structural hot mix asphalt concrete.

6.1 Summary of Results

With respect to the first objective of effects of manufactured fines content on the behaviour of Type 72 mix, the following observations can be made:

- Increasing the level of manufactured fine aggregate in the Type 72 mix resulted in improved densification properties in the gyratory compactor, with the mix passing the Superpave™ specifications at $N_{initial}$ of less than 89 percent of G_{mm} when the manufactured fines content was increased to 60 percent (mean percent G_{mm} of 88.9 percent).

- Marshall stability increased significantly as the proportion of manufactured fines was increased from 20 to 40 (increase of 22 percent), and to 60 percent (increase of 36 percent).
- Marshall flow for the Type 72 mix with 60 percent manufactured fines was 28 percent higher than for the Type 72 mix with 20 percent manufactured fines.
- Dynamic modulus increased across frequency and deviatoric stress state when the amount of manufactured fines was increased from 20 to 60 percent (increase of 51 to 52 percent increase at 10 Hz, and of 24 to 27 percent at 0.5 Hz, across stress state). There was no significant difference in dynamic modulus between 20 and 40 percent manufactured fines content.
- The recoverable axial microstrains were reduced across frequency and deviatoric stress state when the manufactured fines content was increased to 60 percent (reduction of 32 to 33 percent at 10 Hz, and of 18 to 20 percent at 0.5 Hz, across stress state). There was no significant difference between the RAMS at 20 and 40 percent manufactured fines.
- The recoverable radial microstrains were reduced when the manufactured fines content was increased from 20 to 60 percent at the high loading frequency (reduction of 18 to 22 percent at 10 Hz across stress state). There was no significant difference between the RRMS at 20 and 40 percent manufactured fines content.
- Poisson's ratio increased at the high loading frequency of 10 Hz and at the deviatoric stress states of 425 kPa (20 percent increase) and 500 kPa (23 percent increase) when the manufactured fines content was increased from 20 to 60 percent. There was no significant difference between Poisson's ratio at 20 and 40 percent manufactured fines content.
- The phase angle increased across frequency and deviatoric stress state when

the manufactured fines content was raised to 60 percent (increase of 26 to 28 percent at 10 Hz, and 19 to 24 percent at 0.5 Hz, across stress state). There was no significant change in phase angle between 20 and 40 percent manufactured fines.

With respect to the second objective, of comparing the mechanistic behaviour of Type 72 mixes at various levels of manufactured fines to SDHT Type 70 structural mix, the following observations can be made:

- Type 70 mix failed the Superpave™ $N_{initial}$ densification criterion (with mean percent of G_{mm} of 90.1 percent), along with the Type 72 mix at 20 and at 40 percent manufactured fines content. Failure to meet the $N_{initial}$ criterion indicates that the mixes may prove problematic during construction (i.e. tender mixes) and may be susceptible to permanent deformation.

- Marshall stability results for mix Type 70 (which had 38 percent of manufactured fines) were 22 percent higher than the Marshall stability for the Type 72 mix with 20 percent manufactured fines. The Type 70 Marshall stability results were statistically the same as those for the Type 72 mix with 40 percent manufactured fines, with a mean of 10,069 N, compared to a mean of 10,084N, respectively. Type 70 mix had lower Marshall stability than Type 72 mix with 60 percent manufactured fines, whose mean Marshall stability was 11.181 N.

- There was no difference in Marshall flow between the Type 70 mix and the Type 72 mixes with 20 and with 40 percent manufactured fines, respectively. Type 70 mix (with a mean of 1.9 mm) had lower Marshall flow than Type 72 mix with 60 percent manufactured fines (with a mean of 2.3 mm).

- Dynamic modulus for Mix Type 70 (with means of 1950 to 2317 MPa at 10 Hz, and 958 to 1200 MPa at 0.5 Hz) was the same as that for Mix Type 72 at 20 and at 40 percent manufactured fines, respectively, and was lower than the dynamic modulus for Type 72 mix with 60 percent manufactured fines (with means of 2784 to 3292 MPa at 10 Hz, and 1159 to 1489 MPa at 0.5 Hz).

- Recoverable axial microstrains for Type 70 mix (means ranging across deviatoric stress state from 255 to 300 x 10^{-6} at 10 Hz, and 501 to 625 x 10^{-6} at 0.5 Hz) were statistically the same across deviatoric stress state and frequency as the RAMS for Type 72 mix with 20 and 40 percent manufactured fines, respectively. Type 70 mix resulted in RAMS higher than the Type 72 mix with 60 percent manufactured fines. At the highest deviatoric stress state of 500 kPa the Type 70 mix resulted in RAMS lower than the Type 72 mix at 20 and at 40 percent manufactured fines content, respectively.

- The recoverable radial microstrains for the Type 70 mix (with means ranging from 82 to 86 x 10^{-6} at 10 Hz, and from 218 to 279 x 10^{-6} at 0.5 Hz) are statistically the same as the RRMS for the Type 72 mix with 20, 40, and 60 percent manufactured fines, respectively, at each stress state and frequency.

- There was no significant change in Poisson's ratio between the Type 70 mix (with means ranging from 0.29 to 0.32 at 10 Hz, and 0.43 to 0.45 at 0.5 Hz) and the Type 72 mixes at 20 and at 40 percent manufactured fines content, respectively. Type 70 mix resulted in a lower Poisson's ratio than the Type 72 mix with 60 percent manufactured fines (means ranging from 0.36 to 0.38 at 10 Hz, and 0.47 to 0.49 at 0.5 Hz).

- Type 70 mix phase angle (with means ranging from 21.3 to 22.0 degrees at 10 Hz, and 19 to 20.7 degrees at 0.5 Hz) resulted in a phase angle lower than Type 72 mix with 60 percent manufactured fines content (with means ranging from 26 to 27.4 degrees at 10 Hz, and from 22.1 to 24.0 degrees at 0.5 Hz). There was no significant change in phase angle between the Type 70 mix and the Type 72 mixes at 20 and at 40 percent manufactured fines content, respectively.

With respect to the economic implications of implementing finer mixes, the following observations can be made:

- Assuming SDHT continues to re-surface approximately 500 km of asphalt pavement roads annually, engineering rut resistant mixes has the potential to result in $102.5 million savings in asphalt concrete pavement life cycle costs over the next 18 years, at which point the entire pavement network will have been rehabilitated with the improved asphalt mixes.

- Manufacturing 12.5 mm top size mixes results in an estimated decrease of 38 to 58 percent in the amount of rejected pea gravel, therefore optimizing gravel source utilization.

- Manufacturing 12.5 mm top size mixes results in an approximate increase of three to eight percent in the volume of manufactured material that can be produced from a gravel source, resulting in 18,000 to 48,000 metric tonnes of aggregate savings annually. Over the next 42 years, the potential savings amount to 2,016,000 tonnes. The monetary value of the aggregate saved over the next 42 years is in the order of $1.7 million in present day dollars.

- Improving the level of pavement performance results in reduced needs for preservation treatments, therefore decreasing the need for aggregate during the life cycle of an asphalt concrete pavement road. Approximately 53,772 metric tonnes of aggregate can be saved on an annual basis, and there is a potential to save 2,258,424 metric tonnes between 2007 and 2049. The monetary value of these aggregate savings is in the order of $19.1 million.

- The total potential cost savings after 18 years of paving 500 km per year with rut-resistant, well-performing HMAC mixes amount to $112.4 million in present value dollars and to $193.7 million over the next 42 years.

- The total potential aggregate savings that can be realized by implementing well-performing Type 72 HMAC mixes amount to 4.3 million metric tonnes of aggregate. These savings could compensate for approximately six percent of the short fall in provincial aggregate needs up to the year 2049.

Additional observations that can be made after completing this research project include:

- Superpave™ compaction protocols result in higher densification of the SDHT mixes than the standard Marshall compaction. The 75 blow Marshall compaction resulted in mean VTM ranging from 3.9 to 4.2 percent for the research mixes, compared to 3.1 to 3.6 percent VTM after N_{design} level of compaction in the gyratory compactor. Also, at $N_{maximum}$ the gyratory samples for each of the research mixes compacted below the SDHT acceptable VTM level of three percent (mean percent G_{mm} ranging from 97.4 to 97.8 percent).

- Strict adherence to laboratory procedures has an impact on the level of confidence that can be obtained in test results. The increased coefficient of variation in the VTM of the gyratory samples for Type 72 mix with 40 percent manufactured fines (CV of 12 percent) may have been caused by variations in laboratory procedure. For instance, it is possible that there was variability in the time period between sample preparation and compaction, which could have affected the amount of cooling in the HMAC prior to compaction.

- There was a significant amount of variability for Marshall flow results within the ten repeat samples for each mix type, with coefficients of variation ranging from 11 to 18 percent, depending on the mix type.

- Based on the ten repeat Marshall samples compacted by SDHT laboratory for use in this research project, the level of confidence of two repeat samples ranged from 65 to 75 percent for Marshall VTM, stability, and flow, depending on mix type. If these measurements are used to apply penalties and make decisions to accept or reject field HMAC, a higher number of repeat samples is recommended, to increase the level of confidence in the results. For example, using five repeat Marshall samples would increase the level of confidence to 90 percent based on VTM.

- Loading frequency highly influences the magnitude of the mechanistic properties measured, for each of the research mixes. Specifically, the magnitude of dynamic modulus reduced by 50 percent when axial loading frequency was lowered from 10 to 0.5 Hz. Similarly, RAMS increased by approximately 50 percent, RRMS increased by approximately 100 percent, and Poisson's ratio increased by approximately 25 percent when the axial loading frequency was lowered to 0.5 Hz.
- The increase in phase angle for the Type 72 mix with 60 percent manufactured fines indicates that the stiffness of the asphalt binder is being mobilized in mix response to loading. This is likely related to the fact that the mix now contains more fractured surfaces, which tend to bond better with asphalt cement, resulting in a delayed response to loading.
- Because it is a ratio of two parameters, Poisson's ratio can be insensitive to mix response to changes in stress state and loading frequency; therefore axial and radial microstrains should be characterized individually to better quantify mix strain and deformation behaviour.

6.2 Conclusions

The research hypothesis stated that increasing amount of manufactured fines improves mechanistic properties of the Type 72 mix under typical field state conditions, and Type 72 mix with increased manufactured fines can exhibit mechanistic properties equivalent to or exceeding those of a typical type 70 mix.

Based on the improved densification properties, increased Marshall stability, increased dynamic modulus, and reduced radial and axial strains, it is apparent that increasing manufactured fines content in the Type 72 mix does improve the mechanistic properties of this dense-graded asphalt mix. It should be noted that there appears to be a minimum level of manufactured fines content that is required to affect mix response to loading, and that this threshold lies somewhere between 40 and 60 percent manufactured fines content as a portion of total fine aggregate for the Type 72 mix tested as part of this research.

Further, the Type 72 mix exhibited comparable or improved mechanistic properties relative to the Type 70 mix, which SDHT considers a structural mix. This is illustrated by the Type 72 mix with 60 percent manufactured fines resulting in higher Marshall stability and dynamic modulus, and lower axial microstrains than the Type 70 mix evaluated in this study.

Economic analysis indicates that substantial savings in life cycle costs of SHDT asphalt concrete surfaced roadways can be realized by engineering well-performing, rut-resistant mixes. Further, enhanced crushing of smaller aggregate top size decreases the amount of rejected material, and increases manufactured fines to coarse aggregate ratio, resulting not only in better engineering properties, but also in the optimized use of the province's diminishing gravel resources. Pressures on aggregate are also reduced by improving life cycle performance of Saskatchewan asphalt concrete pavements. The total potential aggregate savings that can be realized by implementing well-performing Type 72 HMAC mixes amount to 4.3 million metric tonnes of aggregate in the next 42 years. These aggregate savings can help decrease the predicted shortage of aggregate between 2007 and 2049 by approximately 6 percent. The total potential cost savings based on reduction in pavement life cycle costs after 18 years of paving 500 km per year with rut-resistant, well-performing HMAC mixes, amount to $112.4 million in present value dollars. The 42 year savings amount to $193.7 million in present day dollars.

6.3 Future Research

This research resulted in important findings that, if implemented, may significantly reduce the life cycle costs of SDHT asphalt concrete surfaced roads. The economic evaluation included as part of this study should be expanded upon, and should address not only the financial implications for Saskatchewan Highways and Transportation budgets, but also the many benefits to the road users and to the provincial economy as a whole.

The findings of this research are based on laboratory characterization, and are limited to the testing protocol used and to the asphalt mix types tested, as well as their physical properties. It is recommended that when possible, other Type 72 and Type 70 mixes are evaluated using similar testing protocols. In addition, field test sections should be used to further verify the research hypothesis investigated here. Specifically, the following future research would be useful to further quantify SDHT mixes and their sensitivity to manufactured aggregate content:

- Compare the influence of manufactured fines content on the Type 72 mix used in this research to a coarser Type 70 mix than was available for this research project.
- Compare the laboratory mechanistic characterization of manufactured fines content in SDHT asphalt mixes to their field performance through constructing test sections of the various research mixes evaluated as part of this research.
- Compare the laboratory mechanistic characterization of manufactured fines content in SDHT asphalt mixes to field mechanistic structural measures.
- Evaluate the influence of manufactured fines content on different Type 72 mixes, and compare to the Type 72 mix used in this research.
- Evaluate the influence of manufactured fines content on Type 70 and Type 71 mixes.

LIST OF REFERENCES

AASHTO TP4. 1995. Standard Method for Preparing and Determining the Density of Hot-Mix Asphalt Specimens by Means of the Superpave^{TM} Gyratory Compactor. American Association of State Highway and Transportation Officials Provisional Standards.

Ahlrich, R.C. 1996. Influence of Aggregate Properties on Performance of Heavy-Duty Hot-Mix Asphalt Pavements. Transportation Research Record No. 1547, pp. 7-14.

Allen, D.A., and Haisler, W.E. 1985. Introduction to Aerospace Structural Analysis. John Wiley & Sons, New York, New York.

Anderson, R.M., Bukowski, J.R., Turner, P.A. 1999. Using Superpave Performance Tests to Evaluate Asphalt Mixtures. Transportation Research Record No. 1681, pp. 106-112.

Anderson, T. 2005. Senior Traffic Engineer, Saskatchewan Highways and Transportation. Personal Correspondence.

Anthony, A.M., and Berthelot, C.F. 2003. Rutting Performance of SHRP SPS-9A Test Site in Saskatchewan. Proceedings of the 48th Annual Conference, Canadian Technical Asphalt Association, Vol. XLVIII, pp. 377-398.

Aschenbrener T., and MacKean, C. 1994. Factors that Affect the Voids in the Mineral Aggregate of Hot-Mix Asphalt. Transportation Research Record No. 1469, pp.1-8.

Asphalt Institute. 1983. Principles of Construction of Hot-Mix Asphalt Pavements (MS-22). College Park, Maryland.

Asphalt Institute. 1996. SuperpaveTM Mix Design (SP-2). Lexington, Kentucky.

Asphalt Institute. 1997. Mix Design Methods for Asphalt Concrete and Other Hot-Mix Types (MS-2) 6th Edition. Lexington, Kentucky.

Asphalt Institute. 1998. Construction of Hot Mix Asphalt Pavements (MS-22) 2nd Edition. Lexington, Kentucky.

AASHTO T 245. Resistance to Plastic Flow of Bituminous Mixtures using Marshall Apparatus. American Association of State Highway and Transportation Officials Provisional Standards.

AASHTO T 246. Resistance to Deformation and Cohesion of Bituminous Mixtures by Means of Hveem Apparatus. American Association of State Highway and Transportation Officials Provisional Standards.

AASHTO T 283. Resistance of Compacted Bituminous Mixture to Moisture-Induced Damage. American Association of State Highway and Transportation Officials Provisional Standards.

ASTM C 1252. Standard Method for Uncompacted Void Content of Fine Aggregate – Method A. Annual Book of ASTM Standards, American Society for Testing and Materials.

ASTM D 1559. Standard Test Method for Resistance to Plastic Flow for Bituminous Mixtures Using Marshall Apparatus. Annual Book of ASTM Standards, American Society for Testing and Materials.

ASTM D 1560. Standard Test Methods for Resistance to Deformation and Cohesion of Bituminous Mixtures by Means of Hveem Apparatus. Annual Book of ASTM Standards, American Society for Testing and Materials.

ASTM D 2419. Standard Test Method for Sand Equivalent Value of Soils and Fine Aggregate. Annual Book of ASTM Standards, American Society for Testing and Materials.

ASTM D 2726. Standard Test Method for Bulk Specific Gravity and Density of Non-Absorptive Compacted Bituminous Mixtures. Annual Book of ASTM Standards, American Society for Testing and Materials.

ASTM D 2844. Standard Test Method for Resistance R-Value and Expansion Pressure of Compacted Soils. Annual Book of ASTM Standards, American Society for Testing and Materials.

ASTM D 3497. Standard Test Method for Dynamic Modulus of Asphalt Mixtures. Annual Book of ASTM Standards, American Society for Testing and Materials.

ASTM D 4318. Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils. Annual Book of ASTM Standards, American Society for Testing and Materials.

ASTM D 4791. Standard Test Method for Flat or Elongated Particles in Coarse Aggregate. Annual Book of ASTM Standards, American Society for Testing and Materials.

Baumgartner, E. 2005. Triaxial Frequency Sweep Characterization of Dense Graded Hot Mix Asphalt Concrete Mixes. M.Sc. Thesis, Department of Civil Engineering, University of Saskatchewan.

Berthelot, C.F., Crockford, B., Lytton, R. 1999. Comparison of Alternative Test Methods for Predicting Asphalt Concrete Rut Performance. Proceedings of the 44th Annual Conference, Canadian Technical Asphalt Association, Vol. XLVL, pp. 405-434.

Berthelot, C.F. 1999. Mechanistic Modeling of Saskatchewan SPS-9A Asphalt Concrete Pavements. Ph.D. Dissertation, Department of Civil Engineering, University of Saskatchewan.

Berthelot, C.F., and Gerbrandt, R. 2002. Cold In Place Recycling and Full Depth Strengthening of Expansive Subgrade Soils using Cementitious Waste Products in Northern Climates. Transportation Research Record No. 1787, pp. 3-12.

Berthelot, C.F., Allen, D., Searcy, C. 2003. Method for Performing Accelerated Characterization of Viscoelastic Constitutive Behaviour of Asphalt Concrete. Journal of Materials in Civil Engineering. American Society of Civil Engineers. Vol. 15, no. 5, pp. 496-505.

Berthelot, C.F., Gehlen, T., Scullion, T., Drever, D. 2005. Use of Non-Destructive Testing in the Structural Rehabilitation Decisions of a Primary Urban Corridor. Proceedings of 84th Annual Meeting of the Transportation Research Board, Paper No. 05-0960.

Berthelot, C.F., Scullion, T., Luhr, D. 2005. Investigation of Cement Modification of Granular Base and Subbase Materials using Triaxial Frequency Sweep Characterization. Transportation Research Board, 84th Annual Conference. CD-ROM Proceedings, Paper No. 05-1371.

Berthelot, C.F., Anthony, A.M., Raducanu, C. 2005. Mechanistic Characterization of Anti-Stripping Additives in Saskatchewan Asphalt Mixes. Annual Conference of the Transportation Association of Canada.

Berthelot, C.F., Marjerison, B., Houston, G., McCaig, J., Warrener, S., Gorlick, R. 2007. Mechanistic Comparison of Cementitious and Bituminous Stabilized Granular Base Systems. Transportation Resarch Board, 85th Annual Conference. CD-ROM Proceedings, (Accepted).

Beshara, M. 2002. Rutting Assessment on CS 1-20 and CS 1-16. Internal Report, Saskatchewan Highways and Transportation.

Beshara, M. 2004. Evaluation of Lime versus Liquid Additive as Anti-Stripping Agents. Internal Memorandum, Saskatchewan Highways and Transportation.

Bray, Kerry. 2006. Laboratory Manager, Saskatchewan Highways and Transportation. Personal Correspondence.

Brown, R.R., and Bassett, C.E. 1990. Effects of Maximum Aggregate Size on Rutting Potential and Other Properties of Asphalt –Aggregate Mixtures. Transportation Research Record 1259, pp. 107-119.

Brown, E.R., and Cross, S.A. 1992. A National Study of Rutting in Hot Mix Asphalt (HMA) Pavements. NCAT Report 92-5, National Centre for Asphalt Technology.

Brown, E.R., Kandhal, P.S., Zhang, J. 2004. Performance Testing for Hot-Mix Asphalt. New Simple Performance Tests for Asphalt Mixes, Transportation Research Circular No. E-C068, Transportation Research Board, Washington, DC. pp. 85-106.

Brule, B. 1996. Polymer-Modified Asphalt Cements Used in the Road Construction Industry: Basic Principles. Transportation Research Record No. 1535, pp. 48-53.

Button, J.W., Perdomo, D., Lytton, R.L. 1990. Influence of Aggregate on Rutting in Asphalt Concrete Pavements. Transportation Research Record No. 1259, pp.141-152.

Carlberg, M. 2003. The Effect of Coarse Aggregate Angularity on Rutting Performance of Saskatchewan Asphalt Concrete Mixes. M.Sc. Thesis, Department of Civil Engineering, University of Saskatchewan.

Carlberg, M., Berthelot, C.F., Richardson, N. 2002. In-Service Performance Investigation of Saskatchewan Highways and Transportation Asphalt Concrete Mixes. Proceedings of the 47th Annual Conference, Canadian Technical Asphalt Association, Vol. XLVII, pp. 153-174.

Carpenter, S.H., and Vavrik, W.R. 2001. Repeated Triaxial Testing During Mix Design for Performance Characterization. Transportation Research Record No. 1767, pp.76-84.

Cominsky, R.J., Huber, G.A., Kennedy, T.W., Anderson, M. 1994. The Superpave Mix Design Manual for New Construction and Overlays. SHRP-A-407. Strategic Highway Research Program, National Research Council, Washington, D.C.

Coree, B.J. 1999. Hot-Mix Asphalt Volumetrics Revisited – A New Paradigm. Transportation Research Record No. 1681, pp. 50-58.

Crockford, W.W., Berthelot, C., Tritt, B., Sinadinos, C. 2002. Rapid Triaxial Test. Asphalt Paving Technology: Association of Asphalt Paving Technologists – Proceedings of the Technical Sessions, v 71, 2002, pp. 712-724.

Czarnecki, A.M., Berthelot, C.F., White, S., Gerbrandt, A.R. 1999. Radisson Specific Pavement Studies – 9A Test Site. Canadian Society for Civil Engineers – Proceedings of Annual Conference, v IV, pp.399-408.

Davis, R.L., 1995. Engineering Properties of Asphalt Mixtures and Their Relationship to Performance. Properties of Asphalt Mixtures and the Relationship to their Performance, ASTM STP 1265, Editors Huber, G.A. and Decker, D.S., American Society for Testing and Materials, Philadelphia.

Duczek, D. 1987. Rutting Analysis of Full Depth Asphalt Mix. Report prepared for Annual Conference of the Western Association of Canadian Highway Officials. Saskatchewan Highways and Transportation.

Emery, J. 1990. Asphalt Pavement Rutting Experience in Canada. Proceedings of the 35th Annual Conference, Canadian Technical Asphalt Association, Vol. XXXV, pp. 80-91.

FHWA. 2000. Validation of Asphalt Binder and Mixture Tests That Measure Rutting Susceptibility. FHWA-RD-99-204, Federal Highway Administration, McLean, Virginia.

Field, F. 1958. The Importance of Percent Crushed in Coarse Aggregates as Applied to Bituminous Pavements. Asphalt Paving Technology: Association of Asphalt Paving Technologists – Proceedings of the Technical Sessions, v 27, 1958, pp. 295-322.

Foster, C.R. 1993. Development of Marshall Procedures for Designing Asphalt Paving Mixtures. National Asphalt Pavement Association, Lanham, Maryland.

Frass, L. 2007. Asset Management Engineer, Saskatchewan Highways and Transportation. Personal Correspondence.

Goyal, V.K., and Johnson, E.R. 2003. Cohesive-Decohesive Interfacial Constitutive Law for the Analyses of Fatigue Crack Initiation and Growth. Collection of Technical Papers – 44th AIAA/ASME/ASCE/AHS/ASC Structures Dynamics and Materials Conference, American Institute of Aeronautics and Astronautics Inc. v 4, pp. 2514-2524.

Haddock, J., Pan, C., Feng, A., White, T.D. 1999. Effect of Gradation on Asphalt Mixture Performance. Transportation Research Record No. 1681, pp. 59-68.

Hafez, I.H., and Witczak, M.W. 1995. Comparison of Marshall and Superpave Level I Mix Design for Asphalt Mixes. Transportation Research Record No. 1492, pp.161-175.

Halldorson, Terry. 2007. Gravel Source Manager, Saskatchewan Highways and Transportation, Personal Correspondence.

Hand, A.J., and Epps., A.L. 2001. Impact of Gradation Relative to Superpave Restricted Zone on Hot-Mix Asphalt Performance. Transportation Research Record No. 1767, pp. 158-166.

Hesp, S.A, Smith, B.J., Hoare, T.R. 2002. Effect of Filler Particle Size on Low and High Temperature Performance in Asphalt Mastic and Concrete. Asphalt Paving Technology: Association of Asphalt Paving Technologists – Proceedings of the Technical Sessions, v 70, 2002, pp. 492-508.

Holtz, R.D., and Kovacs, W.D. 1981. An Introduction to Geotechnical Engineering. Editors Newmark, N.M. and Hall, W.J., Prentice-Hall, Englewood Cliffs, New Jersey.

Huber, G.A., and Heiman, G.H. 1986. Effect of Asphalt Concrete Parameters on Rutting Performance: A Field Investigation. Saskatchewan Highways and Transportation.

Huber, G.A., and Heiman, G.H. 1989. Effect of Asphalt Concrete Parameters on Rutting Performance: A Field Investigation. Pavement Rutting Seminar- Roads and Transportation Association of Canada.

Huber, G.A. 1996. Development of the Superpave Gyratory Compactor. The Superpave Asphalt Research Program, The Univeristy of Texas in Austin, web article, http://www.utexas.edu/research/superpave/articles/gyr_hist.html.

Huber, G.A, Jones, J.C., Messersmith, P.E., Jackson, N.M. 1998. Contribution of Fine Aggregate Angularity and Particle Shape to Superpave Mixture Performance. Transportation Research Record No. 1609, pp. 28-35.

Kalyar, I., 2006. Preservation Planner, Saskatchewan Highways and Transportation. Personal Correspondence.

Kandhal, P.S., Brown, E.R. 1990. Comparative Evaluation of 4-inch and 6-inch Diameter Specimens for Testing Large Stone Asphalt Mixes. NCAT Report No. 90-5, National Center for Asphalt Technology, Auburn University, Alabama.

Kandhal, P.S., Motter, J.B., Khatri, M.A. 1991. Evaluation of Particle Shape and Texture: Manufactured versus Natural Sands. NCAT Report No. 91-3, National Center for Asphalt Technology, Auburn University, Alabama.

Kandhal, P.S., and Chakraborty, S. 1996. Evaluation of the Voids in the Mineral Aggregate for HMA Paving Mixtures. NCAT Report No. 96-4, National Center for Asphalt Technology, Auburn University, Alabama.

Kandhal, P.S., Wu, Y., Parker, F.Jr., Spellerberg, P.A. 1996. Precision of Marshall Stability and Flow Tests Using 6-in. (154-mm) Diameter Specimens. Journal of Testing and Evaluation, JTEVA, Vol. 24, No. 1, pp. 20-25.

Kandhal, P.S., and Mallick, R.B. 2001. Effect of Mix Gradation on Rutting Potential of Dense-Graded Asphalt Mixtures. Transportation Research Record No. 1767, pp. 146-151.

Kandhal, P.S. and Cooley, A. Jr. 2002. Coarse- Versus Fine-Graded Superpave Mixtures. Transportation Research Record No. 1789, pp. 216-224.

Kennedy, T.W., Roberts, F.L., Lee, K.W. 1983. Evaluation of Moisture Effects on Asphalt Concrete Mixtures. Transportation Research Record, pp. 134-143.

Kennedy, T.W., Huber, G.A., Harrigan, E.T., Cominsky, R.J., Hughes, C.S., von Quintus, H.,Moulthrop, J.S. 1994. Superior Performing Asphalt Pavements (Superpave): The Product of the SHRP Asphalt Research Program. SHRP-A-410, Strategic Highway Research Program, National Research Council, Washington D.C.

Kuennen, T. 2000. ARTBA's Research on Industry Growth. Better Roads. Better Roads, January, Vol. 70, No. 1. pp. 12-14.

Kuennen, T. 2005. Polymer-Modified Asphalt Comes of Age. Better Roads, November, 2005.

Kuennen, T. 2003-A. A New Era for Permeable Pavements. Better Roads, April, Vol. 73, No. 4. pp. 28-32.

Kuennen, T. 2003-B. Stone Matrix Asphalt is Catching on in the U.S.. Better Roads, September, Vol. 73, No. 9. pp. 22-27.

Kwon, D. 2007. Capital Programming Engineer, Saskatchewan Highways and Transportation. Personal Correspondence.

Lim, C.-T., Tan, S.-A., Fwa, T.-F. 1995. Specimen Size Effects on the Diametrical Mechanical Testing of Cylindrical Asphalt Mixes. Journal of Testing and Evaluation, JTEVA, Vol 23, No. 6, pp. 436-441.

Linden, R.N., Mahoney, J.P., Jackson, N.C. 1989. Effect of Compaction on Asphalt Concrete Performance. Transportation Research Record No. 1217, pp. 20-28.

Marjerison, B. 2005. Materials Engineer, Saskatchewan Highways and Transportation. Personal Correspondence.

Marks, V.J., Monroe, R.W., Adam, J.F. 1990. Effects of Crushed Particles in Asphalt Mixtures. Transportation Research Record No. 1259, pp. 91-106.

McBagonluri, F., Akpan, E., Mercer, C., Shen, W., Soboyejo, W.O. 2005. An Investigation of the Effects of Microstructure on Dwell Fatigue Crack Growth in Ti-6242. Materials Science and Engineering Journal, A – Structural Materials, v 405, n 1-2, pp. 111-134.

NCHRP. 2004. Simple Performance Tester for Superpave Mix Design. Project 9-29, National Cooperative Highway Research Program, Transportation Research Board of National Academies, Washington, DC.

NCHRP. 2005. Superpave Support and Performance Models Management. Project 9-19, National Cooperative Highway Research Program, Transportation Research Board of National Academies, Washington, DC.

NAPA. 2001. HMA Pavement Mix Type Selection Guide. Information Series 128, National Asphalt Pavement Association.

Parker, F. and Brown, E.R. 1992. Effects of Aggregate Properties on Flexible Pavement Rutting in Alabama. Effects of Aggregates and Mineral Fillers on Asphalt Mixture Performance, ASTM Special Technical Publication 1147, R.C. Meininger, Ed., American Society for Testing and Materials, Philadelphia, Pennsylvania, pp. 68-89.

Pellinen, T.K. and Witczak, M.W. 2002. Use of Stiffness of Hot-Mix Asphalt as a Simple Performance Test. Transportation Research Record No. 1789, pp. 80-90.

Perdomo, D., Button, J.W., Lytton, R.L. 1992. A New Approach for Prediction of Permanent Deformation, ASTM Special Technical Publication 1147, R.C. Meininger, Ed., American Society for Testing and Materials, Philadelphia, Pennsylvania, pp. 295-309.

Peterson, W. 2005. Preservation Services Engineer, Saskatchewan Highways and Transportation. Personal Correspondence.

Ponniah, J., and Kennepohl, G. 1996. Polymer-Modified Asphalt Pavements in Ontario: Performance and Cost Effectiveness. Transportation Research Record No. 1545, pp. 151-160.

Prowell, B.D. 2001. Design, Construction and early Performance of Virginia's Hot-Mix Asphalt Stabilizer and Modifier Test Sections. Transportation Research Record No. 1767, pp. 7-14.

Prowell, B.D., Scherocman, J.A., Kennedy, R. 2005. Comparison of Rutting Potential Resulting from Different Levels of Coarse and Fine Aggregate Angularity. Proceedings of 84th Annual Meeting of the Transportation Research Board, Paper No. 05-2535.

Roadbuliders Saskatchewan. 2005. Move on Saskatchewan! – Or Why We Should Invest Now in Saskatchewan's Roads & Transport System. Roadbuilders & Heavy Construction Association of Saskatchewan. Electronic Publication.

Roberts, F.L., Kandhal, P.S., Brown, E.R., Lee, D., Kennedy, T.W. 1996. Hot Mix Asphalt Materials, Mixture Design and Construction, 2nd Edition. NAPA Education Foundation, Lanham, Maryland.

Root R. E. 1989. The Effects of Testing and Production Procedures on Mix Design Results. Proceedings of the 34th Annual Conference, Canadian Technical Asphalt Association, Vol. XXXIV, pp. 116-125.

Sanders, C.A., and Dukatz, E.L. 1992. Evaluation of Percent Fracture of Hot Mix Asphalt Gravels in Indiana. Effects of Aggregates and Mineral Fillers on Asphalt Mixture Performance, ASTM Special Technical Publication 1147, R.C. Meininger, Ed., American Society for Testing and Materials, Philadelphia, Pennsylvania, pp. 90-103.

Sauer, E. K. 2001. CE 864 Introduction to Terrain Analysis and Air Photo Interpretation Class Notes. University of Saskatchewan.

SDHT. 1999. Parallel Process on Roads – Saskatchewan Road Impact Analysis. Saskatchewan Highways and Transportation.

SDHT. 2001-A. Aggregate Management Strategy Review. Draft Final Report, Project Services Group, Saskatchewan Highways and Transportation.

SDHT. 2001-B. Interim Surfacing Manual, Saskatchewan Highways and Transportation.

SDHT. 2003-A. Summary Report on Rutting in Flexible Pavement (Asphalt) in Saskatchewan. Geotechnical and Surfacing Team, Saskatchewan Highways and Transportation.

SDHT. 2003-B. 4100 - Specification for Asphalt Concrete. Standard Specifications Manual, Saskatchewan Highways and Transportation.

SDHT. 2006. 2005-2006 Annual Report. Saskatchewan Highways and Transportation.

SGI. 2002. Traffic Accident Information System (TAIS) Summary. http://www.sgi.sk.ca//sgi_internet/sgi_pub/road_safety/trafficcollisionstats/TAIS2002summary.htm, Saskatchewan Government Insurance.

Shenoy, A., and Romero, P. 2002. Standardized Procedure for Analysis of Dynamic Modulus [E*] Data to Predict Asphalt Pavement Distresses. Transportation Research Record No. 1789, pp. 173-182.

SHRP. 1993. Development and Validation of Performance Prediction Models and Specifications for Asphalt Binders and Paving Mixtures. SHRP-A-357, Strategic Highway Research Program, National Research Council, Washington, D.C.

SHRP. 1994-A. Stage 1 Validation of the Relationship Between Asphalt Properties and Asphalt-Aggregate Mix Performance. SHRP-A-398, Strategic Highway Research Program, National Research Council, Washington, D.C.

SHRP. 1994-B. Permanent Deformation Response of Asphalt Aggregate Mixes. SHRP-A-415, Strategic Highway Research Program, National research Council, Washington, D.C.

Siciliano, J., and Qayyum, A. 1994. Evaluation of Popcorn Mix Performance on the Regina By-Pass CS 1-09(B). Saskatchewan Highways and Transportation.

Sotil, A., Kaloush, K.E, Witczak, M.W. 2004. Proceedings of 83rd Annual Meeting of the Transportation Research Board, Paper No. 04-5180.

Sousa J.B., Craus J., Monismith C.L. 1991. Summary Report on Permanent Deformation in Asphalt Concrete. Strategic Highways Research Program IR-91-104.

Sousa J.B., Way G., Harvey J.T., Hines M. 1995. Comparison of Mix Design Concepts. Transportation Research Record No. 1492, pp. 151-160.

STP 204-10. Marshall Mix Design. Specifications for Standard Test Procedures Manual, Saskatchewan Highways and Transportation.

STP 204-21. Density and Void Characteristics. Specifications for Standard Test Procedures Manual, Saskatchewan Highways and Transportation.

STP 206-04. Atterberg Plasticity Index. Specifications for Standard Test Procedures Manual, Saskatchewan Highways and Transportation.

STP 206-05. Sand Equivalent. Specifications for Standard Test Procedures Manual, Saskatchewan Highways and Transportation.

STP 206-07. Specific Gravity. Specifications for Standard Test Procedures Manual, Saskatchewan Highways and Transportation.

STP 206-09. Lightweight Pieces. Specifications for Standard Test Procedures Manual, Saskatchewan Highways and Transportation.

STP 206-14. Percent Fractured Faces in Aggregate. Specifications for Standard Test Procedures Manual, Saskatchewan Highways and Transportation.

STP 206-15. Stripping Potential and Splitting Tensile Strength. Specifications for Standard Test Procedures Manual, Saskatchewan Highways and Transportation.

Sullivan, III, M. 2004. Statistics: Informed Decisions Using Data. Pearson Education, Inc., Upper Saddle River, New Jersey.

Tayebali, A.A., Malpass, G.A., Khosla, N.P. 1998. Effect of Mineral Filler Type and Amount on Design and Performance of Asphalt Concrete Mixtures. Transportation Research Record No. 1609, pp. 38-43.

Yoder, E.J., and Witczak, M.W. 1975. Principles of Pavement Design, Second Edition. Wiley-Interscience, New York, New York.

Wedding, P.A., and Gaynor, R.D. 1961. The Effects of Using Crushed Gravel as the Coarse and Fine Aggregate in Dense-Graded Bituminous Mixtures. Asphalt Paving Technnology: Association of Asphalt Paving Technologists – Proceedings of the Technical Sessions, v 30, pp. 469.492.

Weissman, S.L., Harvey, J., Sackman, J.L., Long, F. 1999. Selection of Laboratory Test Specimen Dimension for Permanent Deformation of Asphalt Concrete Pavements. Transportation Research Record 1681, pp. 113-120.

Widger, A. 2005. Executive Director, Engineering Standards, Saskatchewan Highways and Transportation. Personal Correspondence.

Witczak, M.W., Kaloush, K.E., Von Quintus, H. 2002. Pursuit of the Simple Performance Test for Asphalt Mixture Rutting. Asphalt Paving Technology: Association of Asphalt Paving Technologists – Proceedings of the Technical Sessions, v 71, 2002, pp. 671-691.

Zubeck, H. 2003. Workability and Performance of Polymer-Modified Asphalt Aggregate Mixtures in Cold Regions. International Journal of Pavement Engineering, v. 4, n. 1, March 2003, pp. 25-36.

APPENDIX A. SDHT MIX DESIGN SUMMARY SHEETS FOR RESEARCH MIXES

Saskatchewan Highways and Transportation

QUALITY CONTROL SYSTEM
MARSHALL DESIGN

Contract #		Pit File or Land Loc.		Date	
Stockpile#	1	Mix Design 1-20	Agg.Type 72	Control Sec.	Type 72 (20%MF)
Limits of Job					
District	80	Contractor		Sublet	
Refinery	HUSKY LLOYD	Pen of Asphalt in Reclaim			
Engineering		Type of Asphalt added	150-200a		
		Type of Asphalt in Mix	150-200a		

AGGREGATE GRADATION

	Stockpile Average % Passing							
		Crushed		Filler	Reclaimed		Design	Spec.
Commodity	Nat Fine	Coarse	Fine	Blender	Mix	Lime	Mix	Limits
Proportion	0.550	0.310	0.140				1.000	
CDN Metric Sieve Series					(AGG)			
18mm	100.0	100.0	100.0	100.0			100.0	
16mm	100.0	100.0	100.0	100.0			100.0	
12.5mm	100.0	96.2	100.0	100.0			98.8	
9mm	100.0	49.6	100.0	99.6			84.4	
5mm	95.1	2.2	87.8	98.4			65.3	+,- 5.0
2mm	72.4	0.9	52.2	95.6			47.4	+,- 4.0
900um	48.2	0.8	35.5	73.0			31.7	+,- 3.0
400um	25.1	0.7	23.6	17.2			17.3	+,- 3.0
160um	10.6	0.6	14.0	2.7			8.0	+,- 2.0
71um	3.8	0.5	8.8	0.3			3.5	+,- 1.5
Sand Equiv.	57.6		77.0				60.7	
% Fracture							95.2	

MARSHALL PROPERTIES

	Test results		Desireable	Max. Theoretical Specific Gravity	
Property	50 Blow	75 Blow	Results	% Asphalt	T.S.G.
Density		2372		5.1	2.494
Air voids		4.2	3.0% - 5.0%	5.2	2.491
V.M.A.		14.9	Minimum 14.5%	5.3	2.488
% Voids Filled		71.6	Maximum 78%	5.5	2.481
Stability		8244.5	Minimum 7000n	5.6	2.478
Flow		1.8	Minimum 2	5.7	2.475
% Stab. retained			Minimum 70%	5.8	2.472
Film Thickness	9.04		Minimum 7.5≅m	5.9	2.468
% Asphalt Absorbed	0.54			6	2.465
Dust Proportion	0.80			6.1	2.462
Flat & Elongated Agg.	3.99		% Manuf'd Fines	19.9	
Fine Angularity	41.86		Plasticity Index	NP	
			Lightweight Pieces	0.2	

Bulk Spec. Grav. Aggregate	2.652	Rice Correction	0.030
DESIGN ASPHALT CONTENT	5.4	TSG	2.484
Asphalt Content of Reclaim		New Asphalt Added to Mix	5.40
Anti-stripping Agent Content		Product	
Anti-stripping Agent Content		Product	
Anti-stripping Agent Content		Product	
Mixing Temperature	143 C	Compaction Temperature	133 C
COMMENTS			

Figure A.1 SDHT Marshall Mix Design Summary for Type 72(20%MF)

Saskatchewan Highways and Transportation

QUALITY CONTROL SYSTEM
MARSHALL DESIGN

Contract #	na-40%mfir	Pit File or Land Loc.					Date	
Stockpile#	1	Mix Design	1-40	Agg.Type	72	Control Sec.	Type 72 (40%MF)	
Limits of Job								
District	80	Contractor				Sublet		
Refinery	HUSKY LLOYD		Pen of Asphalt in Reclaim					
Engineering			Type of Asphalt added			150-200a		
			Type of Asphalt in Mix			150-200a		

AGGREGATE GRADATION

	Stockpile Average % Passing							
		Crushed		Filler	Reclaimed		Design	Spec.
Commodity	Nat Fine	Coarse	Fine	Blender	Mix	Lime	Mix	Limits
Proportion	0.400	0.310	0.290				1.000	
CDN Metric Sieve Series					(AGG)			
18mm	100.0	100.0	100.0	100.0			100.0	
16mm	100.0	100.0	100.0	100.0			100.0	
12.5mm	100.0	96.2	100.0	100.0			98.8	
9mm	100.0	49.6	100.0	99.6			84.4	
5mm	95.1	2.2	87.8	98.4			64.2	+,- 5.0
2mm	72.4	0.9	52.2	95.6			44.4	+,- 4.0
900um	48.2	0.8	35.5	73.0			29.8	+,- 3.0
400um	25.1	0.7	23.6	17.2			17.1	+,- 3.0
160um	10.6	0.6	14.0	2.7			8.5	+,- 2.0
71um	3.8	0.5	8.8	0.3			4.2	+,- 1.5
Sand Equiv.	57.6		77.0				64.7	
% Fracture							97.0	

MARSHALL PROPERTIES

	Test results		Desireable	Max. Theoretical Specific Gravity	
Property	50 Blow	75 Blow	Results	% Asphalt	T.S.G.
Density		2385.9		5.1	2.504
Air voids		4.1	3.0% - 5.0%	5.2	2.501
V.M.A.		14.6	Minimum 14.5%	5.3	2.498
% Voids Filled		72.2	Maximum 75%	5.4	2.494
Stability		10083.9	Minimum 7000n	5.6	2.488
Flow		1.9	Minimum 2	5.7	2.485
% Stab. retained			Minimum 70%	5.8	2.482
Film Thickness	8.53		Minimum 7.5≅m	5.9	2.478
% Asphalt Absorbed	0.62			6	2.475
Dust Proportion	0.97			6.1	2.472
Flat & Elongated Agg.	4.43		% Manuf'd Fines	40.7	
Fine Angularity	42.89		Plasticity Index	NP	
			Lightweight Pieces	0.3	

Bulk Spec. Grav. Aggregate	2.659	Rice Correction	0.035		
DESIGN ASPHALT CONTENT	5.4	TSG	2.494		
Asphalt Content of Reclaim		New Asphalt Added to Mix		5.40	
Anti-stripping Agent Content		Product			
Anti-stripping Agent Content		Product			
Anti-stripping Agent Content		Product			
Mixing Temperature	143 C	Compaction Temperature		133 C	
COMMENTS					

Figure A.2 SDHT Marshall Mix Design Summary for Type 72(40%MF)

Saskatchewan
Highways and
Transportation

QUALITY CONTROL SYSTEM
MARSHALL DESIGN

Contract #		Pit File or Land Loc.		Date	
Stockpile#	1	Mix Design	1	Agg.Type 72	Control Sec. Type 72 (60%MF)
Limits of Job					
District	80	Contractor		Sublet	
Refinery	HUSKY LLOYD	Pen of Asphalt in Reclaim			
Engineering		Type of Asphalt added	150-200a		
		Type of Asphalt in Mix	150-200a		

AGGREGATE GRADATION

	Stockpile Average % Passing							
		Crushed		Filler	Reclaimed		Design	Spec.
Commodity	Nat Fine	Coarse	Fine	Blender	Mix	Lime	Mix	Limits
Proportion	0.265	0.310	0.425				1.000	
CDN Metric Sieve Series					(AGG)			
18mm	100.0	100.0	100.0	100.0			100.0	
16mm	100.0	100.0	100.0	100.0			100.0	
12.5mm	100.0	96.2	100.0	100.0			98.8	
9mm	100.0	49.6	100.0	99.6			84.4	
5mm	95.1	2.2	87.8	98.4			63.2	+,- 5.0
2mm	72.4	0.9	52.2	95.6			41.7	+,- 4.0
900um	48.2	0.8	35.5	73.0			28.1	+,- 3.0
400um	25.1	0.7	23.6	17.2			16.9	+,- 3.0
160um	10.6	0.6	14.0	2.7			8.9	+,- 2.0
71um	3.8	0.5	8.8	0.3			4.9	+,- 1.5
Sand Equiv.	57.6		77.0				68.4	
% Fracture							97.8	

MARSHALL PROPERTIES

	Test results		Desireable	Max. Theoretical Specific Gravity	
Property	50 Blow	75 Blow	Results	% Asphalt	T.S.G.
Density		2395.6		5.1	2.512
Air voids		4.0	3.0% - 5.0%	5.2	2.508
V.M.A.		14.4	Minimum 14.5%	5.3	2.505
% Voids Filled		72.2	Maximum 78%	5.4	2.502
Stability		11180.9	Minimum 7000n	5.6	2.495
Flow		2.3	Minimum 2	5.7	2.492
% Stab. retained			Minimum 70%	5.8	2.489
Film Thickness	8.15		Minimum 7.5ℵm	5.9	2.486
% Asphalt Absorbed	0.66			6	2.483
Dust Proportion	1.13			6.1	2.479
Flat & Elongated Agg.	5.20		% Manuf'd Fines	60.1	
Fine Angularity	45.05		Plasticity Index	NP	
			Lightweight Pieces	0.2	

Bulk Spec. Grav. Aggregate	2.664	Rice Correction	0.038	
DESIGN ASPHALT CONTENT	5.4	TSG	2.502	
Asphalt Content of Reclaim		New Asphalt Added to Mix		5.40
Anti-stripping Agent Content		Product		
Anti-stripping Agent Content		Product		
Anti-stripping Agent Content		Product		
Mixing Temperature	143 C	Compaction Temperature		133 C
COMMENTS				

Figure A.3 SDHT Marshall Mix Design Summary for Type 72(60%MF)

Saskatchewan Highways and Transportation

QUALITY CONTROL SYSTEM
MARSHALL DESIGN

Contract #	TY-70	Pit File or Land Loc.			Date		
Stockpile#	1	Mix Design	1	Agg.Type	70	Control Sec.	Type 70 (38%MF)
Limits of Job							
District	80	Contractor			Sublet		
Refinery	HUSKY LLOYD		Pen of Asphalt in Reclaim				
Engineering			Type of Asphalt added		150-200a		
			Type of Asphalt in Mix		150-200a		

AGGREGATE GRADATION

	Stockpile Average % Passing							
		Crushed		Filler	Reclaimed		Design	Spec.
Commodity	Nat Fine	Coarse	Fine	Blender	Mix	Lime	Mix	Limits
Proportion	0.410	0.340	0.250				1.000	
CDN Metric Sieve Series					(AGG)			
18mm	100.0	100.0	100.0	100.0			100.0	
16mm	100.0	87.7	100.0	100.0			95.8	
12.5mm	100.0	61.5	100.0	100.0			86.9	
9mm	100.0	28.6	100.0	99.6			75.7	
5mm	95.1	3.8	87.8	98.4			62.2	+,- 5.0
2mm	72.4	1.0	52.2	95.6			43.1	+,- 4.0
900um	48.2	0.9	35.5	73.0			28.9	+,- 3.0
400um	25.1	0.8	23.6	17.2			16.5	+,- 3.0
160um	10.6	0.7	14.0	2.7			8.1	+,- 2.0
71um	3.8	0.6	8.8	0.3			4.0	+,- 1.5
Sand Equiv.	57.6		77.0				63.2	
% Fracture							90.7	

MARSHALL PROPERTIES

	Test results		Desireable	Max. Theoretical Specific Gravity	
Property	50 Blow	75 Blow	Results	% Asphalt	T.S.G.
Density				5.1	2.511
Air voids			3.0% - 5.0%	5.2	2.508
V.M.A.			Minimum 14.%	5.3	2.505
% Voids Filled			Maximum 75%	5.5	2.498
Stability			Minimum 7000n	5.6	2.495
Flow			Minimum 2	5.7	2.492
% Stab. retained			Minimum 70%	5.8	2.489
Film Thickness	8.71		Minimum 7.5μm	5.9	2.486
% Asphalt Absorbed	0.72			6	2.482
Dust Proportion	0.93			6.1	2.479
Flat & Elongated Agg.	2.09		% Manuf'd Fines	37.3	
Fine Angularity	42.42		Plasticity Index	NP	
Retained Tens Strgth			Lightweight Pieces	0.2	

Bulk Spec. Grav. Aggregate	2.660	Rice Correction	0.041		
DESIGN ASPHALT CONTENT	5.4	TSG	2.502		
Asphalt Content of Reclaim		New Asphalt Added to Mix		5.40	
Anti-stripping Agent Content	0.7%	Product			
Anti-stripping Agent Content		Product			
Anti-stripping Agent Content		Product			
Mixing Temperature	143 C	Compaction Temperature		133 C	
COMMENTS					

Figure A.4 SDHT Marshall Mix Design Summary for Type 70(38%MF)

APPENDIX B. PHYSICAL PROPERTIES OF AGGREGATES IN RESEARCH MIXES

Table B.1 Sand Equivalent Determination for Mix Type 72(20%MF)

	Sample 1	**Sample 2**	**Sample 3**
Sand Height	51	61	46
Clay Height	74	83	71
Sand Equivalent	69	73	65
Mean Sand Equivalent		**69**	

Table B.2 Sand Equivalent Determination for Mix Type 72(40%MF)

	Sample 1	**Sample 2**	**Sample 3**
Sand Height	38	57	41
Clay Height	52	73	59
Sand Equivalent	73	78	69
Mean Sand Equivalent		**74**	

Table B.3 Sand Equivalent Determination for Mix Type 72(60%MF)

	Sample 1	**Sample 2**	**Sample 3**
Sand Height	43	54	42
Clay Height	59	71	60
Sand Equivalent	73	76	70
Mean Sand Equivalent		**73**	

Table B.4 Sand Equivalent Determination for Mix Type 70(38%MF)

	Sample 1	**Sample 2**	**Sample 3**
Sand Height	51	61	46
Clay Height	74	83	71
Sand Equivalent	69	73	65
Mean Sand Equivalent		**69**	

Table B.5 Uncompacted Voids Determination for Mix Type 72(20%MF)

	Sample 1	Sample 2
Weight Agg. And Measure	306	305.9
Weight Measure	152.6	152.6
Weight Aggregate	153.4	153.3
Volume of Measure	99.45	99.45
BSG of Fine Aggregate	2.653	2.653
Percent Uncompacted Voids	41.8	41.9
Mean Uncompacted Voids	**41.9**	

Table B.6 Uncompacted Voids Determination for Mix Type 72(40%MF)

	Sample 1	Sample 2	Sample 3
Weight Agg. And Measure	303.5	303.7	303.9
Weight Measure	152.6	152.6	152.6
Weight Aggregate	150.9	151.1	151.3
Volume of Measure	99.45	99.45	99.5
BSG of Fine Aggregate	2.659	2.659	2.7
Percent Uncompacted Voids	42.9	42.9	42.8
Mean Uncompacted Voids		**42.9**	

Table B.7 Uncompacted Voids Determination for Mix Type 72(60%MF)

	Sample 1	Sample 2
Weight Agg. And Measure	298	298.4
Weight Measure	152.6	152.6
Weight Aggregate	145.4	145.8
Volume of Measure	99.45	99.45
BSG of Fine Aggregate	2.665	2.665
Percent Uncompacted Voids	45.1	45.0
Mean Uncompacted Voids	**45.1**	

Table B.8 Uncompacted Voids Determination for Mix Type 70(38%MF)

	Sample 1	Sample 2
Weight Agg. And Measure	304.9	305
Weight Measure	152.6	152.6
Weight Aggregate	152.3	152.4
Volume of Measure	99.45	99.45
BSG of Fine Aggregate	2.660	2.660
Percent Uncompacted Voids	42.4	42.4
Mean Uncompacted Voids	**42.4**	

Table B.9 Percent Fracture Determination for Mix Type 72(20%MF)

	Sample 1	Sample 2
Weight of Fractured Aggregate	406.8	411.9
Total Weight of Sample	425.6	434
Percent Fracture	95.6	94.9
Mean Percent Fracture	**95.2**	

Table B.10 Percent Fracture Determination for Mix Type 72(40%MF)

	Sample 1	Sample 2	Sample 3
Weight of Fractured Aggregate	413.5	428.6	433.4
Total Weight of Sample	425.8	442.5	448.6
Percent Fracture	97.1	96.9	96.6
Mean Percent Fracture		**96.9**	

Table B.11 Percent Fracture Determination for Mix Type 72(60%MF)

	Sample 1	Sample 2
Weight of Fractured Aggregate	428.3	450.1
Total Weight of Sample	438.8	459.3
Percent Fracture	97.6	98.0
Mean Percent Fracture	**97.8**	

Table B.12 Percent Fracture Determination for Mix Type 70(38%MF)

	Sample 1	Sample 2
Weight of Fractured Aggregate	403.1	427.1
Total Weight of Sample	451.6	463.7
Percent Fracture	89.3	92.1
Mean Percent Fracture	**90.7**	

Table B.13 Percent Flat and Elongated Pieces for Mix Type 72(20%MF)

	Sample 1
Total Weight of Sample	434
Weight of Flat Pieces	14.7
Percent of Flat Pieces	3.4
Weight of Elongated Pieces	2.6
Percent of Elongated Pieces	0.6
Percent Flat & Elongated Pieces	**4.0**

Table B.14 Percent Flat and Elongated Pieces for Mix Type 72(40%MF)

	Sample 1	Sample 2	Sample 3
Total Weight of Sample	442.5	436.1	442.5
Weight of Flat Pieces	17.2	14.1	16.0
Percent of Flat Pieces	3.9	3.2	3.6
Weight of Elongated Pieces	2.4	4.6	3.9
Percent of Elongated Pieces	0.5	1.1	0.9
Percent Flat & Elongated Pieces	4.4	4.3	4.5
Mean Percent Flat & Elongated Pieces		**4.4**	

Table B.15 Percent Flat and Elongated Pieces for Mix Type 72(60%MF)

	Sample 1
Total Weight of Sample	457.3
Weight of Flat Pieces	20.7
Percent of Flat Pieces	4.5
Weight of Elongated Pieces	3.1
Percent of Elongated Pieces	0.7
Percent Flat & Elongated Pieces	**5.2**

Table B.16 Percent Flat and Elongated Pieces for Mix Type 70(38%MF)

	Sample 1
Total Weight of Sample	463.2
Weight of Flat Pieces	2.5
Percent of Flat Pieces	0.5
Weight of Elongated Pieces	7.2
Percent of Elongated Pieces	1.6
Percent Flat & Elongated Pieces	**2.1**

Table B.17 Percent Lightweight Pieces for Mix Type 72(20%MF)

	Sample 1	Sample 2	Sample 3
Weight of Lightweight Pieces	1.2	0.3	0.9
Total Weight of Sample	425.6	434	419.5
Percent Lightweight Pieces	0.3	0.1	0.2
Mean Percent Lightweight Pieces		**0.2**	

Table B.18 Percent Lightweight Pieces for Mix Type 72(40%MF)

	Sample 1	Sample 2	Sample 3
Weight of Lightweight Pieces	1.3	0.9	1.1
Total Weight of Sample	425.8	442.5	431.2
Percent Lightweight Pieces	0.3	0.2	0.3
Mean Percent Lightweight Pieces		**0.3**	

Table B.19 Percent Lightweight Pieces for Mix Type 72(60%MF)

	Sample 1	Sample 2	Sample 3
Weight of Lightweight Pieces	1.2	0.3	0.9
Total Weight of Sample	425.6	434	419.5
Percent Lightweight Pieces	0.3	0.1	0.2
Mean Percent Lightweight Pieces		**0.2**	

Table B.20 Percent Lightweight Pieces for Mix Type 70(38%MF)

	Sample 1	Sample 2
Weight of Lightweight Pieces	0.9	1.3
Total Weight of Sample	451.6	463.7
Percent Lightweight Pieces	0.2	0.3
Mean Percent Lightweight Pieces	**0.2**	

APPENDIX C. VOLUMETRIC PROPERTIES OF MARSHALL SAMPLES

Table C.1 Volumetric Properties of Marshall Samples for Mix Type 72(20%MF)

Composition: 31% manufactured coarse, 55% natural fines, 14% manufactured fines
BSG Aggregate = 2.652
% Asphalt = 5.4

Sample Name	Weight in Air (g)	Saturated Surface-Dry Weight (g)	Weight in water (g)	Volume (cm^3)	BSG mix	Density (kg/m^3)	VTM %	VMA %	VFA %
20MF-01	1254.4	1255.0	728.0	527.0	2.380	2373.1	4.2	14.8	71.8
20MF-02	1255.8	1256.4	728.5	527.9	2.379	2371.7	4.2	14.9	71.5
20MF-03	1255.2	1255.9	729.3	526.6	2.384	2376.4	4.1	14.7	72.5
20MF-04	1254.8	1255.3	727.5	527.8	2.377	2370.3	4.3	15.0	71.2
20MF-05	1255.5	1255.9	729.7	526.2	2.386	2378.8	4.0	14.6	72.9
20MF-06	1256.7	1257.2	730.6	526.6	2.386	2379.3	3.9	14.6	73.0
20MF-07	1257.4	1258.2	729.8	528.4	2.380	2372.5	4.2	14.9	71.6
20MF-08	1247.4	1248.4	723.4	525.0	2.376	2368.9	4.4	15.0	70.9
20MF-09	1251.0	1251.5	723.2	528.3	2.368	2360.9	4.7	15.3	69.3
20MF-10	1257.6	1258.0	728.6	529.4	2.376	2368.4	4.4	15.0	70.8
Mean	**1254.6**	**1255.2**	**727.9**	**527.3**	**2.379**	**2372.0**	**4.2**	**14.9**	**71.6**
Std Dev	**3.1**	**3.0**	**2.6**	**1.3**	**0.005**	**5.5**	**0.2**	**0.2**	**1.1**
2 x Std Dev	**6.3**	**6.1**	**5.1**	**2.6**	**0.011**	**11.0**	**0.4**	**0.4**	**2.2**
Variance	**9.9**	**9.2**	**6.6**	**1.6**	**0.000**	**30.1**	**0.0**	**0.0**	**1.2**
CV (%)	**0.3**	**0.2**	**0.4**	**0.2**	**0.2**	**0.2**	**5.2**	**1.3**	**1.5**

Table C.2 Volumetric Properties of Marshall Samples for Mix Type 72(40%MF)

Composition:	31% manufactured coarse, 40% natural fines, 29% manufactured fines
BSG Aggregate =	2.659
% Asphalt =	5.4

Sample Name	Weight in Air (g)	Saturated Surface-Dry Weight (g)	Weight in water (g)	Volume (cm^3)	BSG mix	Density (kg/m^3)	VTM %	VMA %	VFA %
40MF-01	1250.3	1250.8	728.7	522.1	2.395	2387.6	4.0	14.5	72.5
40MF-02	1254.9	1255.2	731.6	523.6	2.397	2389.5	3.9	14.5	72.9
40MF-03	1248.4	1248.7	726.9	521.8	2.392	2385.3	4.1	14.6	72.0
40MF-04	1258.3	1258.8	733.9	524.9	2.397	2390.0	3.9	14.5	73.0
40MF-05	1257.6	1257.9	733.1	524.8	2.396	2389.2	3.9	14.5	72.8
40MF-06	1254.7	1255.3	730.6	524.7	2.391	2384.1	4.1	14.7	71.8
40MF-07	1253.7	1254.4	728.9	525.5	2.386	2378.6	4.4	14.9	70.7
40MF-08	1255.3	1256.0	732.2	523.8	2.397	2389.3	3.9	14.5	72.9
40MF-09	1254.3	1255.0	730.1	524.9	2.390	2382.4	4.2	14.7	71.4
40MF-10	1248.9	1249.4	726.8	522.6	2.390	2382.6	4.2	14.7	71.5
Mean	**1253.6**	**1254.2**	**730.3**	**523.9**	**2.393**	**2385.9**	**4.1**	**14.6**	**72.2**
Std Dev	**3.4**	**3.4**	**2.5**	**1.3**	**0.004**	**3.9**	**0.2**	**0.1**	**0.8**
2 x Std Dev	**6.8**	**6.9**	**4.9**	**2.6**	**0.008**	**7.8**	**0.3**	**0.3**	**1.6**
Variance	**11.6**	**11.8**	**6.1**	**1.7**	**0.000**	**15.0**	**0.0**	**0.0**	**0.6**
CV (%)	**0.3**	**0.3**	**0.3**	**0.3**	**0.2**	**0.2**	**3.8**	**1.0**	**1.1**

Table C.3 Volumetric Properties of Marshall Samples for Mix Type 72(60%MF)

Composition:	31% manufactured coarse, 26.5% natural fines,42.5% manufactured fines
BSG Aggregate =	2.664
% Asphalt =	5.4

Sample Name	Weight in Air (g)	Saturated Surface-Dry Weight (g)	Weight in water (g)	Volume (cm^3)	BSG mix	Density (kg/m^3)	VTM %	VMA %	VFA %
60MF-01	1256.0	1256.4	735.6	520.8	2.412	2404.4	3.6	14.1	74.5
60MF-02	1251.4	1251.8	729.4	522.4	2.395	2388.3	4.2	14.7	71.1
60MF-03	1258.2	1258.8	734.6	524.2	2.400	2393.0	4.1	14.5	72.1
60MF-04	1254.6	1255.3	731.7	523.6	2.396	2388.9	4.2	14.7	71.2
60MF-05	1253.6	1254.0	734.3	519.7	2.412	2404.9	3.6	14.1	74.6
60MF-06	1249.0	1249.4	728.7	520.7	2.399	2391.5	4.1	14.6	71.7
60MF-07	1256.6	1256.8	733.7	523.1	2.402	2395.0	4.0	14.5	72.5
60MF-08	1253.9	1254.2	732.4	521.8	2.403	2395.8	3.9	14.4	72.6
60MF-09	1256.5	1256.9	734.5	522.4	2.405	2398.0	3.9	14.4	73.1
60MF-10	1251.1	1251.3	730.7	520.6	2.403	2396.0	4.4	14.1	68.5
Mean	**1254.1**	**1254.5**	**732.6**	**521.9**	**2.403**	**2395.6**	**4.0**	**14.4**	**72.2**
Std Dev	**2.9**	**2.9**	**2.4**	**1.5**	**0.006**	**5.7**	**0.3**	**0.2**	**1.8**
2 x Std Dev	**5.8**	**5.9**	**4.7**	**2.9**	**0.011**	**11.4**	**0.6**	**0.5**	**3.5**
Variance	**8.4**	**8.6**	**5.6**	**2.1**	**0.000**	**32.6**	**0.1**	**0.1**	**3.1**
CV (%)	**0.2**	**0.2**	**0.3**	**0.3**	**0.2**	**0.2**	**6.9**	**1.6**	**2.5**

Table C.4 Volumetric Properties of Marshall Samples for Mix Type

70(38%MF)

Composition:	34% manufactured coarse, 41% natural fines, 25% manufactured fines
BSG Aggregate =	2.660
% Asphalt =	5.4

Sample Name	Weight in Air (g)	Saturated Surface-Dry Weight (g)	Weight in water (g)	Volume (cm^3)	BSG mix	Density (kg/m^3)	VTM %	VMA %	VFA %
T70-01	1255.0	1255.7	734.3	521.4	2.407	2399.8	3.8	14.2	73.3
T70-02	1255.6	1256.0	733.8	522.2	2.404	2397.2	3.9	14.3	72.8
T70-03	1253.8	1254.1	733.4	520.7	2.408	2400.7	3.7	14.1	73.5
T70-04	1253.7	1254.1	731.9	522.2	2.401	2393.6	4.0	14.4	72.0
T70-05	1251.5	1251.8	730.5	521.3	2.401	2393.5	4.0	14.4	72.0
T70-06	1249.3	1249.6	730.5	519.1	2.407	2399.4	3.8	14.2	73.2
T70-07	1256.1	1256.4	732.6	523.8	2.398	2390.9	4.1	14.5	71.4
T70-08	1252.6	1252.9	731.7	521.2	2.403	2396.1	3.9	14.3	72.5
T70-09	1252.7	1253.0	732.2	520.8	2.405	2398.1	3.8	14.2	72.9
T70-10	1252.9	1253.3	731.6	521.7	2.402	2394.4	4.0	14.4	72.2
Mean	**1253.3**	**1253.7**	**732.3**	**521.4**	**2.404**	**2396.4**	**3.9**	**14.3**	**72.6**
Std Dev	**2.0**	**2.1**	**1.3**	**1.2**	**0.003**	**3.2**	**0.1**	**0.1**	**0.7**
2 x Std Dev	**4.0**	**4.1**	**2.6**	**2.4**	**0.006**	**6.5**	**0.3**	**0.2**	**1.4**
Variance	**4.1**	**4.3**	**1.7**	**1.5**	**0.000**	**10.4**	**0.0**	**0.0**	**0.5**
CV (%)	**0.2**	**0.2**	**0.2**	**0.2**	**0.1**	**0.1**	**3.3**	**0.8**	**0.9**

APPENDIX D. VOLUMETRIC PROPERTIES OF GYRATORY SAMPLES

Table D.1 Correction Factors for Volumetric Properties from Gyratory Compactor for Mix Type 72(20%MF)

Sample No.	G_{mb} (measured @Nmax)	G_{mb} (estimated @Nmax)	Correction Factor
20S01	2.421	2.415	1.003
20S02	2.426	2.413	1.005
20S33	2.426	2.424	1.001
20S04	2.421	2.417	1.001
20S05	2.423	2.413	1.004
20S06	2.428	2.414	1.006
20S07	2.425	2.414	1.004
20S08	2.423	2.404	1.008
20S09	2.415	2.404	1.004
20S10	2.423	2.420	1.001
Mean	**2.423**	**2.414**	**1.004**
Std Dev	**0.004**	**0.006**	**0.002**
2 x Std Dev	**0.007**	**0.012**	**0.004**
Variance	**1.32E-05**	**3.75E-05**	**4.96E-06**
CV (%)	**0.002**	**0.003**	**0.002**

Table D.2 Gyratory Compaction Properties at $N_{initial}$ for Mix Type 72(20%MF)

Composition: 31% manufactured coarse, 55% natural fines, 14% manufactured fines
$BSG_{Aggregate}$ = 2.652
% Asphalt = 5.4

Sample Name	G_{mb} corrected	% G_{mm}	VTM (%)	VTM corrected (%)	VMA (%)	VMA corrected (%)	VFA (%)	VFA corrected (%)
20MFG01	2.238	0.90	10.14	9.91	20.38	20.18	50.23	50.87
20MFG02	2.242	0.92	10.20	9.74	20.43	20.02	50.07	51.36
20MFG33	2.251	0.93	9.46	9.38	19.77	19.70	52.17	52.40
20MFG04	2.239	0.92	10.01	9.88	20.26	20.14	50.60	50.97
20MFG05	2.248	0.93	9.87	9.51	20.14	19.82	50.98	52.03
20MFG06	2.249	0.93	9.97	9.46	20.23	19.77	50.71	52.16
20MFG07	2.240	0.93	10.22	9.82	20.45	20.10	50.02	51.12
20MFG08	2.247	0.93	10.25	9.55	20.47	19.86	49.95	51.90
20MFG09	2.231	0.92	10.60	10.20	20.78	20.43	49.01	50.08
20MFG10	2.243	0.92	9.81	9.71	20.09	20.00	51.16	51.45
Mean	**2.24**	**0.92**	**10.05**	**9.72**	**20.30**	**20.00**	**50.49**	**51.43**
Std Dev	**0.01**	**0.01**	**0.31**	**0.25**	**0.27**	**0.22**	**0.85**	**0.71**
2 x Std Dev	**0.01**	**0.02**	**0.61**	**0.50**	**0.54**	**0.44**	**1.70**	**1.42**
Variance	**0.00**	**0.00**	**0.09**	**0.06**	**0.07**	**0.05**	**0.72**	**0.50**
CV (%)	**0.28**	**0.90**	**3.05**	**2.57**	**1.34**	**1.11**	**1.68**	**1.38**

Table D.3 Gyratory Compaction Properties at N_{design} for Mix Type 72(20%MF)

Composition: 31% manufactured coarse, 55% natural fines, 14% manufactured fines
$BSG_{Aggregate}$ = 2.652
% Asphalt = 5.4

Sample Name	G_{mb} corrected	% G_{mm}	VTM (%)	VTM corrected (%)	VMA (%)	VMA corrected (%)	VFA (%)	VFA corrected (%)
20MFG01	2.399	0.97	3.68	3.44	14.65	14.44	74.88	76.20
20MFG02	2.404	0.97	3.72	3.22	14.69	14.24	74.69	77.41
20MFG33	2.405	0.97	3.27	3.19	14.29	14.22	77.10	77.57
20MFG04	2.399	0.97	3.57	3.43	14.56	14.43	75.46	76.23
20MFG05	2.402	0.97	3.70	3.30	14.67	14.32	74.80	76.93
20MFG06	2.407	0.97	3.65	3.11	14.63	14.14	75.04	78.05
20MFG07	2.403	0.97	3.68	3.25	14.65	14.27	74.90	77.22
20MFG08	2.401	0.97	4.08	3.33	15.01	14.35	72.82	76.76
20MFG09	2.393	0.96	4.10	3.67	15.02	14.65	72.72	74.93
20MFG10	2.401	0.97	3.46	3.35	14.45	14.36	76.09	76.69
Mean	**2.40**	**0.97**	**3.69**	**3.33**	**14.66**	**14.34**	**74.85**	**76.80**
Std Dev	**0.00**	**0.00**	**0.25**	**0.16**	**0.22**	**0.14**	**1.32**	**0.88**
2 x Std Dev	**0.01**	**0.00**	**0.50**	**0.32**	**0.44**	**0.28**	**2.64**	**1.76**
Variance	**0.00**	**0.00**	**0.06**	**0.03**	**0.05**	**0.02**	**1.74**	**0.77**
CV (%)	**0.17**	**0.17**	**6.77**	**4.79**	**1.51**	**0.99**	**1.76**	**1.14**

Table D.4 Gyratory Compaction Properties at $N_{maximum}$ for Mix Type 72(20%MF)

Composition: 31% manufactured coarse, 55% natural fines, 14% manufactured fines

$BSG_{Aggregate}$ = 2.652

% Asphalt = 5.4

Sample Name	G_{mb} corrected	% G_{mm}	VTM (%)	VTM corrected (%)	VMA (%)	VMA corrected (%)	VFA (%)	VFA corrected (%)
20MFG01	2.421	0.97	2.78	2.54	13.86	13.64	79.92	81.41
20MFG02	2.426	0.98	2.84	2.33	13.91	13.46	79.59	82.65
20MFG33	2.426	0.98	2.42	2.33	13.54	13.46	82.12	82.65
20MFG04	2.421	0.97	2.68	2.54	13.77	13.64	80.54	81.41
20MFG05	2.423	0.98	2.85	2.46	13.92	13.57	79.51	81.90
20MFG06	2.428	0.98	2.81	2.25	13.88	13.39	79.79	83.16
20MFG07	2.425	0.98	2.81	2.38	13.88	13.50	79.79	82.40
20MFG08	2.423	0.98	3.21	2.46	14.23	13.57	77.47	81.90
20MFG09	2.415	0.97	3.21	2.78	14.23	13.85	77.47	79.95
20MFG10	2.423	0.98	2.57	2.46	13.67	13.57	81.22	81.90
Mean	**2.42**	**0.98**	**2.82**	**2.45**	**13.89**	**13.57**	**79.74**	**81.93**
Std Dev	**0.00**	**0.00**	**0.25**	**0.15**	**0.22**	**0.13**	**1.45**	**0.90**
2 x Std Dev	**0.01**	**0.00**	**0.49**	**0.29**	**0.44**	**0.26**	**2.90**	**1.80**
Variance	**0.00**	**0.00**	**0.06**	**0.02**	**0.05**	**0.02**	**2.10**	**0.81**
CV (%)	**0.15**	**0.15**	**8.75**	**5.97**	**1.57**	**0.96**	**1.82**	**1.10**

Table D.5 Volumetric Properties By Weight in Water at $N_{maximum}$ for Mix Type 72(20%MF)

Composition: 31% manufactured coarse, 55% natural fines, 14% manufactured fines
$BSG_{Aggregate}$ = 2.652
% Asphalt = 5.4

Sample Name	Weight in Air (g)	SSD Weight (g)	Weight in water (g)	Volume (cm^3)	BSG mix	Density (kg/m^3)	VTM (%)	VMA (%)	VFA (%)
20MFG01	6230.9	6232.6	3658.9	2573.7	2.421	2414	2.6	13.4	80.9
20MFG02	6270.4	6271.8	3686.8	2585	2.426	2418	2.4	13.2	82.1
20MFG33	6268.6	6269.5	3685.1	2584.4	2.426	2418	2.4	13.2	82.1
20MFG04	6269.7	6272.2	3682.7	2589.5	2.421	2414	2.5	13.4	81.0
20MFG05	6270.4	6272.7	3684.9	2587.8	2.423	2416	2.5	13.3	81.5
20MFG06	6271	6272.5	3689.8	2582.7	2.428	2421	2.3	13.1	82.7
20MFG07	6269.6	6271.4	3686	2585.4	2.425	2418	2.4	13.2	82.0
20MFG08	6270.3	6271.8	3684.3	2587.5	2.423	2416	2.5	13.3	81.5
20MFG09	6270.8	6272.5	3675.7	2596.8	2.415	2408	2.8	13.6	79.4
20MFG10	6267.5	6269.3	3682.6	2586.7	2.423	2416	2.5	13.3	81.4
Mean	**6266**	**6268**	**3682**	**2586.0**	**2.42**	**2415.80**	**2.47**	**13.32**	**81.47**
Std Dev	**12.35**	**12.37**	**8.80**	**5.78**	**0.00**	**3.61**	**0.15**	**0.13**	**0.91**
2 x Std Dev	**24.70**	**24.73**	**17.59**	**11.57**	**0.01**	**7.21**	**0.29**	**0.26**	**1.81**
Variance	**152.54**	**152.93**	**77.37**	**33.44**	**0.00**	**13.01**	**0.02**	**0.02**	**0.82**
CV (%)	**0.20**	**0.20**	**0.24**	**0.22**	**0.15**	**0.15**	**5.90**	**0.97**	**1.11**

SSD - Saturated Surface-Dry

Table D.6 Correction Factors for Volumetric Measurements in Gyratory Compactor for Mix Type 72(40%MF)

Sample No.	Gmb (measured @Nmax)	Gmb (estimated @Nmax)	Correction Factor
40S01	2.428	2.421	1.003
40S15	2.440	2.427	1.005
40S03	2.437	2.406	1.013
40S04	2.431	2.417	1.005
40S05	2.408	2.395	1.006
40S06	2.431	2.417	1.006
40S07	2.424	2.423	1.000
40S08	2.444	2.441	1.001
40S09	2.428	2.399	1.012
40S10	2.431	2.418	1.005
Mean	**2.430**	**2.417**	**1.006**
Std Dev	**0.010**	**0.014**	**0.004**
2 x Std Dev	**0.020**	**0.027**	**0.008**
Variance	**9.73E-05**	**1.88E-04**	**1.67E-05**
CV (%)	**0.004**	**0.006**	**0.004**

Table D.7 Gyratory Compaction Properties at $N_{initial}$ for Mix Type 72(40%MF)

Composition: 31% manufactured coarse, 40% natural fines, 29% manufactured fines
$BSG_{Aggregate}$ = 2.659
% Asphalt = 5.4

Sample Name	G_{mb} corrected	% G_{mm}	VTM (%)	VTM corrected (%)	VMA (%)	VMA corrected (%)	VFA (%)	VFA corrected (%)
40MFG01	2.224	0.89	11.05	10.81	21.08	20.86	47.57	48.18
40MFG15	2.236	0.90	10.80	10.33	20.85	20.43	48.21	49.46
40MFG03	2.238	0.90	11.37	10.25	21.36	20.36	46.76	49.67
40MFG04	2.225	0.89	11.31	10.80	21.30	20.86	46.92	48.20
40MFG05	2.196	0.88	12.45	11.96	22.32	21.88	44.21	45.34
40MFG06	2.228	0.89	11.19	10.68	21.20	20.75	47.21	48.52
40MFG07	2.220	0.89	11.01	10.99	21.04	21.02	47.68	47.73
40MFG08	2.245	0.90	10.06	9.97	20.20	20.11	50.18	50.45
40MFG09	2.224	0.89	11.89	10.82	21.82	20.87	45.52	48.16
40MFG10	2.231	0.89	11.01	10.53	21.04	20.61	47.66	48.93
Mean	**2.227**	**0.89**	**11.21**	**10.71**	**21.22**	**20.78**	**47.19**	**48.46**
Std Dev	**0.013**	**0.01**	**0.634**	**0.54**	**0.56**	**0.48**	**1.58**	**1.38**
2 x Std Dev	**0.03**	**0.01**	**1.27**	**1.08**	**1.13**	**0.96**	**3.16**	**1.81**
Variance	**0.0002**	**0.00**	**0.40**	**0.29**	**0.32**	**0.23**	**2.50**	**1.90**
CV (%)	**0.60**	**0.60**	**5.66**	**5.04**	**2.65**	**2.30**	**3.35**	**2.85**

Table D.8 Gyratory Compaction Properties at N_{design} for Mix Type 72(40%MF)

Composition: 31% manufactured coarse, 40% natural fines, 29% manufactured fines
$BSG_{Aggregate}$ = 2.659
% Asphalt = 5.4

Sample Name	G_{mb} corrected	% G_{mm}	VTM (%)	VTM corrected (%)	VMA (%)	VMA corrected (%)	VFA (%)	VFA corrected (%)
40MFG01	2.403	0.96	3.90	3.64	14.73	14.50	73.51	74.88
40MFG15	2.416	0.97	3.63	3.12	14.50	14.04	74.93	77.76
40MFG03	2.412	0.97	4.50	3.29	15.27	14.19	70.50	76.80
40MFG04	2.406	0.96	4.05	3.51	14.87	14.38	72.74	75.61
40MFG05	2.380	0.95	5.09	4.56	15.79	15.32	67.75	70.23
40MFG06	2.406	0.96	4.06	3.51	14.87	14.38	72.71	75.61
40MFG07	2.399	0.96	3.83	3.81	14.67	14.65	73.89	74.00
40MFG08	2.421	0.97	3.04	2.94	13.97	13.88	78.23	78.84
40MFG09	2.404	0.96	4.78	3.63	15.51	14.49	69.19	74.97
40MFG10	2.407	0.96	4.03	3.50	14.84	14.38	72.88	75.64
Mean	**2.405**	**0.96**	**4.09**	**3.55**	**14.90**	**14.42**	**72.63**	**75.43**
Std Dev	**0.011**	**0.00**	**0.584**	**0.44**	**0.52**	**0.39**	**2.97**	**2.32**
2 x Std Dev	**0.02**	**0.01**	**1.17**	**0.88**	**1.04**	**0.78**	**5.94**	**1.81**
Variance	**0.00**	**0.00**	**0.34**	**0.19**	**0.27**	**0.15**	**8.81**	**5.40**
CV (%)	**0.45**	**0.45**	**14.27**	**12.36**	**3.48**	**2.70**	**4.09**	**3.08**

Table D.9 Gyratory Compaction Properties at $N_{maximum}$ for Mix Type 72(40%MF)

Composition: 31% manufactured coarse, 40% natural fines, 29% manufactured fines
$BSG_{Aggregate}$ = 2.659
% Asphalt = 5.4

Sample Name	G_{mb} corrected	% G_{mm}	VTM (%)	VTM corrected (%)	VMA (%)	VMA corrected (%)	VFA (%)	VFA corrected (%)
40MFG01	2.428	0.97	2.91	2.65	13.85	13.62	79.00	80.57
40MFG15	2.440	0.98	2.68	2.17	13.65	13.19	80.35	83.59
40MFG03	2.437	0.98	3.51	2.29	14.38	13.30	75.60	82.81
40MFG04	2.431	0.97	3.07	2.53	14.00	13.51	78.04	81.30
40MFG05	2.408	0.97	3.99	3.45	14.81	14.33	73.08	75.94
40MFG06	2.431	0.97	3.08	2.53	14.00	13.51	78.00	81.30
40MFG07	2.424	0.97	2.83	2.81	13.78	13.76	79.47	79.60
40MFG08	2.444	0.98	2.11	2.00	13.14	13.05	83.94	84.64
40MFG09	2.428	0.97	3.81	2.65	14.65	13.62	73.99	80.57
40MFG10	2.431	0.97	3.05	2.53	13.98	13.51	78.15	81.30
Mean	**2.430**	**0.97**	**3.10**	**2.56**	**14.03**	**13.54**	**77.96**	**81.16**
Std Dev	**0.010**	**0.00**	**0.550**	**0.40**	**0.49**	**0.35**	**3.16**	**2.38**
2 x Std Dev	**0.02**	**0.01**	**1.10**	**0.79**	**0.98**	**0.70**	**6.33**	**1.81**
Variance	**0.00**	**0.00**	**0.30**	**0.16**	**0.24**	**0.12**	**10.00**	**5.69**
CV (%)	**0.41**	**0.41**	**17.72**	**15.46**	**3.48**	**2.59**	**4.06**	**2.94**

Table D.10 Volumetric Properties By Weight in Water at $N_{maximum}$ for Mix Type 72(40%MF)

Composition: 31% manufactured coarse, 40% natural fines, 29% manufactured fines
$BSG_{Aggregate}$ = 2.659
% Asphalt = 5.4

Sample Name	Weight in Air (g)	SSD Weight (g)	Weight in water (g)	Volume (cm^3)	BSG mix	Density (kg/m^3)	VTM (%)	VMA (%)	VFA (%)
40MFG01	6252.7	6254.3	3678.7	2575.6	2.428	2420	2.7	13.4	77.4
40MFG15	6253.6	6255	3691.8	2563.2	2.440	2432	2.2	12.9	80.4
40MFG03	6252.6	6254.5	3688.9	2565.6	2.437	2430	2.3	13.0	79.7
40MFG04	6253.5	6254.9	3682.4	2572.5	2.431	2424	2.5	13.2	78.2
40MFG05	6244.3	6247.7	3654.6	2593.1	2.408	2401	3.5	14.1	73.0
40MFG06	6253.4	6254.7	3682.4	2572.3	2.431	2424	2.5	13.2	78.2
40MFG07	6253.3	6255.2	3675.5	2579.7	2.424	2417	2.8	13.5	76.6
40MFG08	6248.5	6249.6	3693.1	2556.5	2.444	2437	2.0	12.8	81.5
40MFG09	6253.6	6254	3678.2	2575.8	2.428	2421	2.7	13.4	77.5
40MFG10	6253.9	6255.6	3682.9	2572.7	2.431	2424	2.6	13.2	78.2
Mean	**6252**	**6254**	**3681**	**2572.7**	**2.430**	**2422.9**	**2.58**	**13.28**	**81.47**
Std Dev	**3.11**	**2.66**	**10.92**	**9.92**	**0.010**	**9.84**	**0.40**	**0.35**	**0.91**
2 x Std Dev	**6.22**	**5.32**	**21.84**	**19.85**	**0.020**	**19.68**	**0.79**	**0.70**	**1.81**
Variance	**9.66**	**7.07**	**119.23**	**98.50**	**0.000**	**96.79**	**0.16**	**0.12**	**0.82**
CV (%)	**0.05**	**0.04**	**0.30**	**0.39**	**0.406**	**0.41**	**15.34**	**2.65**	**1.11**

SSD - Saturated Surface-Dry

Table D.11 Correction Factors for Volumetric Measurements in Gyratory Compactor for Mix Type 72(60%MF)

Sample No.	Gmb (measured @Nmax)	Gmb (estimated @Nmax)	Correction Factor
60S01	2.438	2.426	1.005
60S02	2.440	2.422	1.007
60S03	2.440	2.412	1.012
60S04	2.452	2.433	1.008
60S05	2.449	2.438	1.005
60S06	2.449	2.436	1.005
60S07	2.435	2.424	1.005
60S08	2.443	2.436	1.003
60S09	2.445	2.432	1.005
60S10	2.442	2.427	1.006
Mean	**2.443**	**2.429**	**1.006**
Std Dev	**0.005**	**0.008**	**0.002**
2 x Std Dev	**0.011**	**0.016**	**0.005**
Variance	**2.93E-05**	**6.44E-05**	**5.85E-06**
CV (%)	**0.002**	**0.003**	**0.002**

Table D.12 Gyratory Compaction Properties at $N_{initial}$ for Mix Type 72(60%MF)

Composition: 31% manufactured coarse, 26.5% natural fines,42.5% manufactured fines
$BSG_{Aggregate}$ = 2.664
% Asphalt = 5.4

Sample Name	G_{mb} corrected	% G_{mm}	VTM (%)	VTM corrected (%)	VMA (%)	VMA corrected (%)	VFA (%)	VFA corrected (%)
60MFG01	2.210	0.88	12.11	11.68	21.91	21.53	44.74	45.75
60MFG02	2.212	0.88	12.23	11.58	22.02	21.44	44.45	45.98
60MFG03	2.214	0.89	12.52	11.49	22.28	21.36	43.80	46.20
60MFG04	2.228	0.89	11.62	10.94	21.48	20.88	45.89	47.58
60MFG05	2.262	0.90	10.00	9.58	20.04	19.67	50.10	51.27
60MFG06	2.231	0.89	11.32	10.85	21.21	20.79	46.64	47.81
60MFG07	2.215	0.89	11.85	11.45	21.68	21.33	45.34	46.30
60MFG08	2.223	0.89	11.41	11.14	21.29	21.05	46.40	47.08
60MFG09	2.222	0.89	11.69	11.21	21.54	21.11	45.72	46.90
60MFG10	2.218	0.89	11.90	11.34	21.73	21.23	45.22	46.58
Mean	**2.224**	**0.89**	**11.67**	**11.13**	**21.52**	**21.04**	**45.83**	**47.15**
Std Dev	**0.015**	**0.01**	**0.692**	**0.61**	**0.61**	**0.54**	**1.73**	**1.60**
2 x Std Dev	**0.03**	**0.01**	**1.38**	**1.21**	**1.23**	**1.08**	**3.47**	**1.81**
Variance	**0.00**	**0.00**	**0.48**	**0.37**	**0.38**	**0.29**	**3.01**	**2.55**
CV (%)	**0.68**	**0.68**	**5.93**	**5.45**	**2.86**	**2.56**	**3.78**	**3.39**

Table D.13 Gyratory Compaction Properties at N_{design} for Mix Type 72(60%MF)

Composition: 31% manufactured coarse, 26.5% natural fines,42.5% manufactured fines
$BSG_{Aggregate}$ = 2.664
% Asphalt = 5.4

Sample Name	G_{mb} corrected	% G_{mm}	VTM (%)	VTM corrected (%)	VMA (%)	VMA corrected (%)	VFA (%)	VFA corrected (%)
60MFG01	2.411	0.96	4.12	3.65	14.81	14.40	72.20	74.63
60MFG02	2.412	0.96	4.29	3.58	14.96	14.33	71.35	75.04
60MFG03	2.413	0.96	4.69	3.57	15.32	14.32	69.41	75.10
60MFG04	2.427	0.97	3.74	3.00	14.48	13.82	74.16	78.29
60MFG05	2.423	0.97	3.60	3.15	14.35	13.95	74.94	77.42
60MFG06	2.421	0.97	3.73	3.23	14.47	14.02	74.20	76.97
60MFG07	2.408	0.96	4.20	3.77	14.89	14.50	71.77	74.01
60MFG08	2.416	0.97	3.71	3.42	14.45	14.19	74.30	75.90
60MFG09	2.419	0.97	3.85	3.33	14.57	14.11	73.58	76.43
60MFG10	2.416	0.97	4.05	3.44	14.75	14.21	72.53	75.78
Mean	**2.42**	**0.97**	**4.00**	**3.41**	**14.70**	**14.19**	**72.84**	**75.96**
Std Dev	**0.01**	**0.00**	**0.34**	**0.24**	**0.30**	**0.21**	**1.71**	**1.33**
2 x Std Dev	**0.01**	**0.00**	**0.67**	**0.48**	**0.60**	**0.43**	**3.42**	**1.81**
Variance	**0.00**	**0.00**	**0.11**	**0.06**	**0.09**	**0.05**	**2.92**	**1.78**
CV (%)	**0.25**	**0.25**	**8.40**	**7.03**	**2.03**	**1.50**	**2.35**	**1.76**

Table D.14 Gyratory Compaction Properties at $N_{maximum}$ for Mix Type 72(60%MF)

Composition: 31% manufactured coarse, 26.5% natural fines,42.5% manufactured fines

$BSG_{Aggregate}$ = 2.664

% Asphalt = 5.4

Sample Name	G_{mb} corrected	% G_{mm}	VTM (%)	VTM corrected (%)	VMA (%)	VMA corrected (%)	VFA (%)	VFA corrected (%)
60MFG01	2.438	0.97	3.03	2.56	13.84	13.43	78.12	80.95
60MFG02	2.440	0.98	3.19	2.48	13.99	13.35	77.17	81.44
60MFG03	2.440	0.98	3.61	2.48	14.36	13.35	74.86	81.44
60MFG04	2.452	0.98	2.75	2.00	13.59	12.93	79.79	84.54
60MFG05	2.449	0.98	2.57	2.12	13.44	13.03	80.88	83.75
60MFG06	2.449	0.98	2.63	2.12	13.49	13.03	80.51	83.75
60MFG07	2.435	0.97	3.12	2.68	13.92	13.53	77.62	80.21
60MFG08	2.443	0.98	2.66	2.36	13.51	13.25	80.34	82.20
60MFG09	2.445	0.98	2.81	2.28	13.65	13.18	79.42	82.71
60MFG10	2.442	0.98	3.02	2.40	13.83	13.28	78.20	81.95
Mean	**2.44**	**0.98**	**2.94**	**2.35**	**13.76**	**13.24**	**78.69**	**82.29**
Std Dev	**0.01**	**0.00**	**0.32**	**0.22**	**0.29**	**0.19**	**1.86**	**1.38**
2 x Std Dev	**0.01**	**0.00**	**0.64**	**0.43**	**0.57**	**0.38**	**3.73**	**1.81**
Variance	**0.00**	**0.00**	**0.10**	**0.05**	**0.08**	**0.04**	**3.47**	**1.91**
CV (%)	**0.22**	**0.22**	**10.92**	**9.23**	**2.07**	**1.45**	**2.37**	**1.68**

Table D.15 Volumetric Properties By Weight in Water at $N_{maximum}$ for Mix Type 72(60%MF)

Composition: 31% manufactured coarse, 26.5% natural fines,42.5% manufactured fines
$BSG_{Aggregate}$ = 2.664
% Asphalt = 5.4

Sample Name	Weight in Air (g)	SSD Weight (g)	Weight in water (g)	Volume (cm^3)	BSG mix	Density (kg/m^3)	VTM (%)	VMA (%)	VFA (%)
60MFG01	6270.7	6272.1	3700.1	2572	2.438	2431	2.5	13.2	80.7
60MFG02	6270.7	6273.1	3703	2570.1	2.440	2433	2.5	13.1	81.1
60MFG03	6271.2	6273.6	3703.8	2569.8	2.440	2433	2.5	13.1	81.3
60MFG04	6254	6256.1	3705.3	2550.8	2.452	2444	2.0	12.7	84.3
60MFG05	6269.1	6270.8	3711.1	2559.7	2.449	2442	2.1	12.8	83.5
60MFG06	6267.5	6269.3	3709.7	2559.6	2.449	2441	2.1	12.8	83.4
60MFG07	6268.8	6270.8	3696.8	2574	2.435	2428	2.7	13.3	80.0
60MFG08	6265.4	6267	3702.7	2564.3	2.443	2436	2.3	13.0	82.0
60MFG09	6265.9	6267.5	3705	2562.5	2.445	2438	2.3	12.9	82.5
60MFG10	6271	6272.1	3704.3	2567.8	2.442	2435	2.4	13.0	81.7
Mean	**6267**	**6269**	**3704**	**2565.1**	**2.443**	**2436.07**	**2.33**	**12.99**	**81.47**
Std Dev	**5.16**	**5.12**	**4.15**	**7.09**	**0.01**	**5.25**	**0.21**	**0.19**	**0.91**
2 x Std Dev	**10.32**	**10.24**	**8.30**	**14.18**	**0.01**	**10.49**	**0.42**	**0.37**	**1.81**
Variance	**26.65**	**26.20**	**17.24**	**50.28**	**0.00**	**27.53**	**0.04**	**0.04**	**0.82**
CV (%)	**0.08**	**0.08**	**0.11**	**0.28**	**0.22**	**0.22**	**9.01**	**1.44**	**1.11**

SSD - Saturated Surface-Dry

Table D.16 Correction Factors for Volumetric Measurements in Gyratory Compactor for Mix Type 70(38%MF)

Sample No.	Gmb (measured @Nmax)	Gmb (estimated @Nmax)	Correction Factor
70S01	2.442	2.434	1.003
70S02	2.442	2.430	1.005
70S03	2.452	2.444	1.003
70S04	2.449	2.437	1.005
70S05	2.464	2.446	1.008
70S06	2.451	2.442	1.004
70S07	2.450	2.446	1.002
70S08	2.445	2.435	1.004
70S09	2.440	2.432	1.003
70S10	2.447	2.448	0.999
Mean	**2.448**	**2.439**	**1.004**
Std Dev	**0.007**	**0.007**	**0.002**
2 x Std Dev	**0.014**	**0.013**	**0.004**
Variance	**4.80E-05**	**4.34E-05**	**4.55E-06**
CV (%)	**0.003**	**0.003**	**0.002**

Table D.17 Gyratory Compaction Properties at $N_{initial}$ for Mix Type 70(38%MF)

Composition: 34% manufactured coarse, 41% natural fines, 25% manufactured fines
$BSG_{Aggregate}$ = 2.660
% Asphalt = 5.4

Sample Name	G_{mb} corrected	% G_{mm}	VTM (%)	VTM corrected (%)	VMA (%)	VMA corrected (%)	VFA (%)	VFA corrected (%)
T70G01	2.241	0.90	10.72	10.44	20.56	20.31	47.84	48.58
T70G02	2.243	0.90	10.79	10.35	20.62	20.23	47.67	48.84
T70G03	2.262	0.90	9.90	9.61	19.83	19.57	50.07	50.89
T70G04	2.258	0.90	10.20	9.76	20.10	19.71	49.23	50.45
T70G05	2.270	0.91	9.94	9.26	19.86	19.26	49.97	51.92
T70G06	2.260	0.90	10.01	9.68	19.93	19.63	49.76	50.71
T70G07	2.262	0.90	9.75	9.60	18.39	19.56	50.49	50.93
T70G08	2.247	0.90	10.54	10.18	20.40	20.08	48.33	49.30
T70G09	2.244	0.90	10.61	10.31	20.46	20.19	48.14	48.95
T70G10	2.253	0.90	9.90	9.96	19.83	19.88	50.07	49.91
Mean	**2.25**	**0.90**	**10.24**	**9.91**	**20.00**	**19.84**	**49.16**	**50.05**
Std Dev	**0.01**	**0.00**	**0.39**	**0.39**	**0.65**	**0.35**	**1.06**	**1.10**
2 x Std Dev	**0.02**	**0.01**	**0.78**	**0.79**	**1.29**	**0.70**	**2.12**	**1.81**
Variance	**0.00**	**0.00**	**0.15**	**0.15**	**0.42**	**0.12**	**1.13**	**1.22**
CV (%)	**0.44**	**0.44**	**3.83**	**3.97**	**3.23**	**1.77**	**2.16**	**2.21**

Table D.18 Gyratory Compaction Properties at N_{design} for Mix Type 70(38%MF)

Composition: 34% manufactured coarse, 41% natural fines, 25% manufactured fines

$BSG_{Aggregate}$ = 2.660

% Asphalt = 5.4

Sample Name	G_{mb} corrected	% G_{mm}	VTM (%)	VTM corrected (%)	VMA (%)	VMA corrected (%)	VFA (%)	VFA corrected (%)
T70G01	2.418	0.97	3.67	3.37	14.29	14.02	74.28	75.94
T70G02	2.418	0.97	3.84	3.36	14.44	14.01	73.37	75.99
T70G03	2.428	0.97	3.26	2.95	13.92	13.64	76.58	78.39
T70G04	2.428	0.97	3.45	2.97	14.08	13.66	75.54	78.25
T70G05	2.442	0.98	3.14	2.41	13.81	13.16	77.30	81.71
T70G06	2.429	0.97	3.26	2.90	13.92	13.60	76.58	78.67
T70G07	2.428	0.97	3.13	2.97	12.41	13.66	77.32	78.27
T70G08	2.420	0.97	3.65	3.26	14.27	13.92	74.42	76.57
T70G09	2.416	0.97	3.76	3.43	14.36	14.07	73.83	75.62
T70G10	2.424	0.97	3.07	3.13	13.75	13.80	77.68	77.34
Mean	**2.43**	**0.97**	**3.42**	**3.08**	**13.93**	**13.76**	**75.69**	**77.67**
Std Dev	**0.01**	**0.00**	**0.29**	**0.31**	**0.58**	**0.27**	**1.61**	**1.82**
2 x Std Dev	**0.02**	**0.01**	**0.58**	**0.62**	**1.17**	**0.55**	**3.21**	**1.81**
Variance	**0.00**	**0.00**	**0.08**	**0.09**	**0.34**	**0.07**	**2.58**	**3.32**
CV (%)	**0.32**	**0.32**	**8.45**	**10.00**	**4.20**	**1.99**	**2.12**	**2.35**

Table D.19 Gyratory Compaction Properties at $N_{maximum}$ for Mix Type 70(38%MF)

Composition: 34% manufactured coarse, 41% natural fines, 25% manufactured fines

$BSG_{Aggregate}$ = 2.660

% Asphalt = 5.4

Sample Name	G_{mb} corrected	% G_{mm}	VTM (%)	VTM corrected (%)	VMA (%)	VMA corrected (%)	VFA (%)	VFA corrected (%)
T70G01	2.442	0.98	2.70	2.40	13.42	13.15	79.87	81.77
T70G02	2.442	0.98	2.88	2.40	13.58	13.15	78.78	81.77
T70G03	2.452	0.98	2.31	2.00	13.08	12.80	82.31	84.38
T70G04	2.449	0.98	2.60	2.12	13.33	12.90	80.52	83.58
T70G05	2.464	0.98	2.25	1.52	13.02	12.37	82.70	87.72
T70G06	2.451	0.98	2.40	2.04	13.16	12.83	81.75	84.12
T70G07	2.450	0.98	2.24	2.08	11.60	12.87	82.77	83.85
T70G08	2.445	0.98	2.67	2.28	13.39	13.05	80.07	82.54
T70G09	2.440	0.98	2.81	2.48	13.52	13.22	79.22	81.26
T70G10	2.447	0.98	2.14	2.20	12.92	12.98	83.45	83.06
Mean	**2.45**	**0.98**	**2.50**	**2.15**	**13.10**	**12.93**	**81.14**	**83.40**
Std Dev	**0.01**	**0.00**	**0.26**	**0.28**	**0.57**	**0.25**	**1.65**	**1.86**
2 x Std Dev	**0.01**	**0.01**	**0.53**	**0.55**	**1.14**	**0.49**	**3.31**	**1.81**
Variance	**0.00**	**0.00**	**0.07**	**0.08**	**0.33**	**0.06**	**2.73**	**3.46**
CV (%)	**0.28**	**0.28**	**10.53**	**12.87**	**4.36**	**1.90**	**2.04**	**2.23**

Table D.20 Volumetric Properties By Weight in Water at $N_{maximum}$ for Mix Type 70(38%MF)

Composition: 34% manufactured coarse, 41% natural fines, 25% manufactured fines
$BSG_{Aggregate}$ = 2.660
% Asphalt = 5.4

Sample Name	Weight in Air (g)	SSD Weight (g)	Weight in water (g)	Volume (cm^3)	BSG mix	Density (kg/m^3)	VTM (%)	VMA (%)	VFA (%)
T70G01	6269.5	6270.6	3703	2567.6	2.442	2434	2.4	12.9	81.5
T70G02	6267.8	6271.6	3705.2	2566.4	2.442	2435	2.4	12.9	81.6
T70G03	6266.3	6267.8	3712.7	2555.1	2.452	2445	2.0	12.5	84.3
T70G04	6270.1	6271.9	3711.6	2560.3	2.449	2442	2.1	12.7	83.4
T70G05	6266.2	6268	3724.9	2543.1	2.464	2457	1.5	12.1	87.6
T70G06	6265.8	6268.1	3711.5	2556.6	2.451	2443	2.0	12.6	83.9
T70G07	6163.7	6165.5	3649.6	2515.9	2.450	2443	2.1	12.6	83.6
T70G08	6268.7	6270.2	3706	2564.2	2.445	2437	2.3	12.8	82.3
T70G09	6268.9	6271.1	3702	2569.1	2.440	2433	2.5	13.0	81.1
T70G10	6267.9	6269.3	3707.4	2561.9	2.447	2439	2.2	12.7	82.8
Mean	**6257**	**6259**	**3703**	**2556.0**	**2.448**	**2440.82**	**2.14**	**12.69**	**81.47**
Std Dev	**32.99**	**33.03**	**20.01**	**16.02**	**0.01**	**6.93**	**0.28**	**0.25**	**0.91**
2 x Std Dev	**65.97**	**66.06**	**40.02**	**32.04**	**0.01**	**13.87**	**0.56**	**0.50**	**1.81**
Variance	**1088.10**	**1091.05**	**400.46**	**256.72**	**0.00**	**48.06**	**0.08**	**0.06**	**0.82**
CV (%)	**0.53**	**0.53**	**0.54**	**0.63**	**0.28**	**0.28**	**13.02**	**1.95**	**1.11**

SSD - Saturated Surface-Dry

APPENDIX E.   MARSHALL STABILITY AND FLOW

Table E.1 Marshall Properties of Mix Type 72 with 20 Percent Manufactured Fines

Sample Name	Marshall Stability (Newton)	Marshall Flow (mm)
20MF-01	8861	1.6
20MF-02	8515	2.3
20MF-03	8918	1.7
20MF-04	8794	1.8
20MF-05	8582	1.3
20MF-06	8371	1.7
20MF-07	8755	1.8
20MF-08	6710	2.4
20MF-09	7027	1.7
20MF-10	7910	1.8
Mean	**8244**	**1.8**
Std Dev	**785**	**0.3**
2 x Std Dev	**1570**	**0.7**
Variance	**616301**	**0.1**
CV (%)	**9.5**	**18.2**

Table E.2 Marshall Properties of Mix Type 72 with 40 Percent Manufactured Fines

Sample Name	Marshall Stability (Newton)	Marshall Flow (mm)
40MF-01	10980	2.0
40MF-02	9859	1.9
40MF-03	9860	1.8
40MF-04	10368	2.0
40MF-05	9792	1.7
40MF-06	10118	2.4
40MF-07	9274	2.0
40MF-08	9794	1.7
40MF-09	9274	1.7
40MF-10	11520	1.7
Mean	**10084**	**1.9**
Std Dev	**709**	**0.2**
2 x Std Dev	**1419**	**0.4**
Variance	**503374**	**0.0**
CV (%)	**7.0**	**11.4**

Table E.3 Marshall Properties of Mix Type 72 with 60 Percent Manufactured Fines

Sample Name	Marshall Stability (Newton)	Marshall Flow (mm)
60MF-01	10820	2.0
60MF-02	11020	2.1
60MF-03	10675	2.3
60MF-04	10406	2.6
60MF-05	10750	2.6
60MF-06	12150	1.9
60MF-07	11347	2.2
60MF-08	11650	2.1
60MF-09	11580	2.8
60MF-10	11410	2.2
Mean	**11181**	**2.3**
Std Dev	**537**	**0.3**
2 x Std Dev	**1074**	**0.6**
Variance	**288448**	**0.1**
CV (%)	**4.8**	**13.5**

Table E.4 Marshall Properties of Mix Type 70 with 38 Percent Manufactured Fines

Sample Name	Marshall Stability (Newton)	Marshall Flow (mm)
T70-01	9840	2.1
T70-02	10240	1.9
T70-03	11300	2.0
T70-04	9840	1.7
T70-05	10780	1.8
T70-06	10000	1.8
T70-07	9706	1.6
T70-08	9000	2.6
T70-09	9820	1.7
T70-10	10160	1.6
Mean	**10069**	**1.9**
Std Dev	**624**	**0.3**
2 x Std Dev	**1248**	**0.6**
Variance	**389430**	**0.1**
CV (%)	**6.2**	**15.9**

APPENDIX F. STANDARD NORMAL DISTRIBUTION PROBABILITY TABLE

Table F.1 Standard Normal Distribution Probability Table

	0.00	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09
0.0	0.0000	0.0040	0.0080	0.0120	0.0160	0.0199	0.0239	0.0279	0.0319	0.0359
0.1	0.0398	0.0438	0.0478	0.0517	0.0557	0.0596	0.0636	0.0675	0.0714	0.0753
0.2	0.0793	0.0832	0.0871	0.0910	0.0948	0.0987	0.1026	0.1064	0.1103	0.1141
0.3	0.1179	0.1217	0.1255	0.1293	0.1331	0.1368	0.1406	0.1443	0.1480	0.1517
0.4	0.1554	0.1591	0.1628	0.1664	0.1700	0.1736	0.1772	0.1808	0.1844	0.1879
0.5	0.1915	0.1950	0.1985	0.2019	0.2054	0.2088	0.2123	0.2157	0.2190	0.2224
0.6	0.2257	0.2291	0.2324	0.2357	0.2389	0.2422	0.2454	0.2486	0.2517	0.2549
0.7	0.2580	0.2611	0.2642	0.2673	0.2704	0.2734	0.2764	0.2794	0.2823	0.2852
0.8	0.2881	0.2910	0.2939	0.2967	0.2995	0.3023	0.3051	0.3078	0.3106	0.3133
0.9	0.3159	0.3186	0.3212	0.3238	0.3264	0.3289	0.3315	0.3340	0.3365	0.3389
1.0	0.3413	0.3438	0.3461	0.3485	0.3508	0.3531	0.3554	0.3577	0.3599	0.3621
1.1	0.3643	0.3665	0.3686	0.3708	0.3729	0.3749	0.3770	0.3790	0.3810	0.3830
1.2	0.3849	0.3869	0.3888	0.3907	0.3925	0.3944	0.3962	0.3980	0.3997	0.4015
1.3	0.4032	0.4049	0.4066	0.4082	0.4099	0.4115	0.4131	0.4147	0.4162	0.4177
1.4	0.4192	0.4207	0.4222	0.4236	0.4251	0.4265	0.4279	0.4292	0.4306	0.4319
1.5	0.4332	0.4345	0.4357	0.4370	0.4382	0.4394	0.4406	0.4418	0.4429	0.4441
1.6	0.4452	0.4463	0.4474	0.4484	0.4495	0.4505	0.4515	0.4525	0.4535	0.4545
1.7	0.4554	0.4564	0.4573	0.4582	0.4591	0.4599	0.4608	0.4616	0.4625	0.4633
1.8	0.4641	0.4649	0.4656	0.4664	0.4671	0.4678	0.4686	0.4693	0.4699	0.4706
1.9	0.4713	0.4719	0.4726	0.4732	0.4738	0.4744	0.4750	0.4756	0.4761	0.4767
2.0	0.4772	0.4778	0.4783	0.4788	0.4793	0.4798	0.4803	0.4808	0.4812	0.4817
2.1	0.4821	0.4826	0.4830	0.4834	0.4838	0.4842	0.4846	0.4850	0.4854	0.4857
2.2	0.4861	0.4864	0.4868	0.4871	0.4875	0.4878	0.4881	0.4884	0.4887	0.4890
2.3	0.4893	0.4896	0.4898	0.4901	0.4904	0.4906	0.4909	0.4911	0.4913	0.4916
2.4	0.4918	0.4920	0.4922	0.4925	0.4927	0.4929	0.4931	0.4932	0.4934	0.4936
2.5	0.4938	0.4940	0.4941	0.4943	0.4945	0.4946	0.4948	0.4949	0.4951	0.4952
2.6	0.4953	0.4955	0.4956	0.4957	0.4959	0.4960	0.4961	0.4962	0.4963	0.4964
2.7	0.4965	0.4966	0.4967	0.4968	0.4969	0.4970	0.4971	0.4972	0.4973	0.4974
2.8	0.4974	0.4975	0.4976	0.4977	0.4977	0.4978	0.4979	0.4979	0.4980	0.4981
2.9	0.4981	0.4982	0.4982	0.4983	0.4984	0.4984	0.4985	0.4985	0.4986	0.4986
3.0	0.4987	0.4987	0.4987	0.4988	0.4988	0.4989	0.4989	0.4989	0.4990	0.4990

APPENDIX G. SAMPLE SIZE ANALYSIS BASED ON CONVENTIONAL CHARACTERIZATION OF THE RESEARCH MIXES

Table G.1 Relationship of Sample Size and Level of Confidence for Marshall Voids in Total Mix across Research Mixes at a Margin of Error of 0.2%

Margin of Error = 0.2 %

Level of Confidence	Z	Sample Size (n)			
		T72(20%MF)	T72(40%MF)	T72(60%MF)	T70(38%MF)
99%	2.58	8	4	13	3
95%	1.96	5	2	7	2
90%	1.65	3	2	5	1
75%	1.15	2	1	3	1
50%	0.68	1	0	1	0

Table G.2 Relationship of Sample Size and Level of Confidence for Marshall Stability across Research Mixes at a Margin of Error of 500 Newton

Margin of Error = 500 Newton

Level of Confidence	Z	Sample Size (n)			
		T72(20%MF)	T72(40%MF)	T72(60%MF)	T70(38%MF)
99%	2.58	16	13	8	10
95%	1.96	9	8	4	6
90%	1.65	7	5	3	4
75%	1.15	3	3	2	2
50%	0.68	1	1	1	1

Table G.3 Relationship of Sample Size and Level of Confidence for Marshall Flow across Research Mixes at a Margin of Error of 0.2 mm

Margin of Error = 0.2 mm

Level of Confidence	Z	Sample Size (n)			
		T72(20%MF)	T72(40%MF)	T72(60%MF)	T70(38%MF)
99%	2.58	18	8	15	15
95%	1.96	10	4	9	8
90%	1.65	7	3	6	6
75%	1.15	4	2	3	3
50%	0.68	1	1	1	1

Table G.4 Relationship of Sample Size and Level of Confidence for Gyratory Voids in Total Mix at N_{design} across Research Mixes at a Margin of Error of 0.2%

Margin of Error = 0.2 %

Level of Confidence	Z	Sample Size (n)			
		T72(20%MF)	T72(40%MF)	T72(60%MF)	T70(38%MF)
99%	2.58	4	32	10	16
95%	1.96	2	18	6	9
90%	1.65	2	13	4	6
75%	1.15	1	6	2	3
50%	0.68	0	2	1	1

APPENDIX H. TRIAXIAL FREQUENCY SWEEP TEST RESULTS AT 20°C

Table H.1 Triaxial Frequency Sweep Test results at 20°C and Deviatoric Stress of 370 kPa for Mix Type 72 with 20 Percent Manufactured Fines

Sample Name	Vertical Traction σ_1 (kPa)	Radial Traction σ_3 (kPa)	Deviatoric Stress σ_D (kPa)	Frequency (Hz)	Dynamic Modulus E_d (MPa)	Phase Angle δ (°)	RAMS	RRMS	Poisson's Ratio, ν
20MFG01	600	230	370	10	2294	19.36	253.8	75.2	0.2963
20MFG01	600	230	370	5	1988	19.17	295.5	97.1	0.3286
20MFG01	600	230	370	1	1430	19.08	418.4	169.1	0.4042
20MFG01	600	230	370	0.5	1266	18.25	472.7	199.4	0.4218
20MFG02	600	230	370	10	2113	21.38	276	89	0.3225
20MFG02	600	230	370	5	1818	20.57	322.9	110.5	0.3422
20MFG02	600	230	370	1	1293	19.43	462	184.6	0.3996
20MFG02	600	230	370	0.5	1153	17.86	519.8	221.3	0.4257
20MFG04	600	230	370	10	2165	21.09	269.2	88.9	0.3302
20MFG04	600	230	370	5	1867	20.53	314.9	108.4	0.3442
20MFG04	600	230	370	1	1341	19.51	446.3	187.9	0.4210
20MFG04	600	230	370	0.5	1202	18.35	498.2	220	0.4416
20MFG05	600	230	370	10	2217	19.93	263.9	92.4	0.3501
20MFG05	600	230	370	5	1838	19.77	319.7	110.9	0.3469
20MFG05	600	230	370	1	1326	19.28	450.5	184.1	0.4087
20MFG05	600	230	370	0.5	1172	17.46	510.4	213.9	0.4191
20MFG06	600	230	370	10	2127	20.7	273.8	85.7	0.3130
20MFG06	600	230	370	5	1847	19.93	317.6	112.3	0.3536
20MFG06	600	230	370	1	1331	19.18	449.4	186	0.4139
20MFG06	600	230	370	0.5	1171	18.03	510.9	219	0.4287
20MFG07	600	230	370	10	2208	19.81	265.2	83.4	0.3145
20MFG07	600	230	370	5	1922	19.43	305.7	104.8	0.3428
20MFG07	600	230	370	1	1353	19.43	442.3	180.8	0.4088
20MFG07	600	230	370	0.5	1197	18.25	499.8	206.4	0.4130
20MFG08	600	230	370	10	2088	20.62	279.2	98.3	0.3521
20MFG08	600	230	370	5	1835	20.59	320.4	129.7	0.4048
20MFG08	600	230	370	1	1311	19.32	456.1	200.1	0.4387
20MFG08	600	230	370	0.5	1172	18	510.7	231.8	0.4539
20MFG09	600	230	370	10	2155	20.73	270.4	87.7	0.3243
20MFG09	600	230	370	5	1883	20.54	312.1	113	0.3621
20MFG09	600	230	370	1	1341	19.46	445.7	185.3	0.4158
20MFG09	600	230	370	0.5	1189	17.92	503.9	217.8	0.4322
20MFG10	600	230	370	10	2124	21.05	274.6	92.6	0.3372
20MFG10	600	230	370	5	1816	20.52	323.9	115.8	0.3575
20MFG10	600	230	370	1	1326	19.07	450.6	194.6	0.4319
20MFG10	600	230	370	0.5	1181	17.55	506.7	223.2	0.4405
20MFG33	600	230	370	10	2180	19.91	267.6	82.6	0.3087
20MFG33	600	230	370	5	1918	20.08	306.4	105.8	0.3453
20MFG33	600	230	370	1	1388	19.27	430.9	176.9	0.4105
20MFG33	600	230	370	0.5	1230	17.48	486.2	207.4	0.4266

Table H.2 Triaxial Frequency Sweep Test results at 20°C and Deviatoric Stress of 370 kPa for Mix Type 72 with 40 Percent Manufactured Fines

Sample Name	Vertical Traction σ_1 (kPa)	Radial Traction σ_3 (kPa)	Deviatoric Stress σ_D (kPa)	Frequency (Hz)	Dynamic Modulus E_d (MPa)	Phase Angle δ (°)	RAMS	RRMS	Poisson's Ratio, ν
40MFG01	600	230	370	10	2316	19.16	251.2	81.8	0.3256
40MFG01	600	230	370	5	1999	19.64	294	101.7	0.3459
40MFG01	600	230	370	1	1423	19.35	420.7	174.4	0.4145
40MFG01	600	230	370	0.5	1249	18.38	479.2	207.7	0.4334
40MFG03	600	230	370	10	2095	20.78	278.2	87.8	0.3156
40MFG03	600	230	370	5	1852	20.44	317.4	112.2	0.3535
40MFG03	600	230	370	1	1309	19.88	456.8	189.4	0.4146
40MFG03	600	230	370	0.5	1149	18.18	520.8	220.4	0.4232
40MFG04	600	230	370	10	2173	20.22	268.1	87.2	0.3253
40MFG04	600	230	370	5	1908	20.71	307.9	114.6	0.3722
40MFG04	600	230	370	1	1329	20.36	450.4	204.2	0.4534
40MFG04	600	230	370	0.5	1154	19.34	518.3	237.5	0.4582
40MFG05	600	230	370	10	2114	20.87	276.2	92.2	0.3338
40MFG05	600	230	370	5	1819	20.19	323	112.1	0.3471
40MFG05	600	230	370	1	1304	19.56	457.9	195.5	0.4269
40MFG05	600	230	370	0.5	1143	18.18	523.8	227.2	0.4338
40MFG06	600	230	370	10	2269	19.91	257.2	84.2	0.3274
40MFG06	600	230	370	5	1953	20.09	300.7	106.2	0.3532
40MFG06	600	230	370	1	1369	20	437.1	185.7	0.4248
40MFG06	600	230	370	0.5	1201	18.69	498.3	221.3	0.4441
40MFG07	600	230	370	10	2137	20.03	274.4	87.4	0.3185
40MFG07	600	230	370	5	1848	20.69	317.7	101.7	0.3201
40MFG07	600	230	370	1	1326	19.79	450.7	194.9	0.4324
40MFG07	600	230	370	0.5	1153	18.63	518.4	224.8	0.4336
40MFG08	600	230	370	10	2203	20.56	264.6	91.9	0.3473
40MFG08	600	230	370	5	1845	21.07	318.6	121.4	0.3810
40MFG08	600	230	370	1	1288	20.04	463.7	204.3	0.4406
40MFG08	600	230	370	0.5	1130	18.81	530.2	248.8	0.4693
40MFG09	600	230	370	10	2098	20.23	277.9	87.8	0.3159
40MFG09	600	230	370	5	1822	19.81	322.6	110.4	0.3422
40MFG09	600	230	370	1	1291	19.28	463.7	185.2	0.3994
40MFG09	600	230	370	0.5	1144	18.05	522.6	219	0.4191
40MFG10	600	230	370	10	2233	20.21	261.6	88	0.3364
40MFG10	600	230	370	5	1913	19.59	307	111.6	0.3635
40MFG10	600	230	370	1	1362	18.41	439	175.8	0.4005
40MFG10	600	230	370	0.5	1190	17.38	503.1	205.9	0.4093
40MFG15	600	230	370	10	2287	19.72	255.6	79.6	0.3114
40MFG15	600	230	370	5	1983	20.53	296.6	94.9	0.3200
40MFG15	600	230	370	1	1398	19.86	427.6	168.6	0.3943
40MFG15	600	230	370	0.5	1219	18.35	491.7	208.4	0.4238

Table H.3 Triaxial Frequency Sweep Test results at 20°C and Deviatoric Stress of 370 kPa for Mix Type 72 with 60 Percent Manufactured Fines

Sample Name	Vertical Traction σ_1 (kPa)	Radial Traction σ_3 (kPa)	Deviatoric Stress σ_D (kPa)	Frequency (Hz)	Dynamic Modulus E_d (MPa)	Phase Angle δ (°)	RAMS	RRMS	Poisson's Ratio, ν
60MFG01	600	230	370	10	3440	27.07	169.4	67.3	0.3973
60MFG01	600	230	370	5	2899	26.66	202.2	87.6	0.4332
60MFG01	600	230	370	1	1811	24.27	330.8	171	0.5169
60MFG01	600	230	370	0.5	1528	22.62	391.9	207.4	0.5292
60MFG02	600	230	370	10	2895	27.58	201.3	80	0.3974
60MFG02	600	230	370	5	2393	27.13	245.3	106.1	0.4325
60MFG02	600	230	370	1	1550	23.7	385.6	193.8	0.5026
60MFG02	600	230	370	0.5	1329	21.67	450.3	229	0.5085
60MFG03	600	230	370	10	3155	28.24	185	73	0.3946
60MFG03	600	230	370	5	2541	27.32	231.2	96.2	0.4161
60MFG03	600	230	370	1	1616	24.67	369.8	170.9	0.4621
60MFG03	600	230	370	0.5	1363	22.2	438.7	209.2	0.4769
60MFG04	600	230	370	10	3232	28.37	180	80	0.4444
60MFG04	600	230	370	5	2652	28.29	221.2	114	0.5154
60MFG04	600	230	370	1	1606	25.15	372.6	197.3	0.5295
60MFG04	600	230	370	0.5	1345	23.1	444.5	239.2	0.5381
60MFG05	600	230	370	10	3949	23.95	147.2	53.7	0.3648
60MFG05	600	230	370	5	3329	23.83	176.2	65.6	0.3723
60MFG05	600	230	370	1	2190	23.87	273.4	126	0.4609
60MFG05	600	230	370	0.5	1826	23.02	327.7	152.2	0.4644
60MFG06	600	230	370	10	4226	23.69	137.6	44.5	0.3234
60MFG06	600	230	370	5	3573	22.95	164.3	59.8	0.3640
60MFG06	600	230	370	1	2334	23.7	256.9	114.9	0.4473
60MFG06	600	230	370	0.5	1941	22.92	308.3	140.8	0.4567
60MFG07	600	230	370	10	2524	16.97	231.1	61.4	0.2657
60MFG07	600	230	370	5	2203	16.76	266.7	76.8	0.2880
60MFG07	600	230	370	1	1615	18.57	370.3	130	0.3511
60MFG07	600	230	370	0.5	1409	18.05	424.7	156.8	0.3692
60MFG08	600	230	370	10	3189	28.02	182.4	72	0.3947
60MFG08	600	230	370	5	2571	27.03	228.2	97.6	0.4277
60MFG08	600	230	370	1	1600	24.78	373.5	174.7	0.4677
60MFG08	600	230	370	0.5	1358	22.56	440.6	211.7	0.4805
60MFG09	600	230	370	10	3293	27.8	176.6	71.6	0.4054
60MFG09	600	230	370	5	2671	27.11	219.7	90.2	0.4106
60MFG09	600	230	370	1	1685	24.97	355.4	168.6	0.4744
60MFG09	600	230	370	0.5	1439	22.79	415.6	213.7	0.5142
60MFG10	600	230	370	10	3016	28.37	192.8	77.5	0.4020
60MFG10	600	230	370	5	2464	27.13	238.1	102.2	0.4292
60MFG10	600	230	370	1	1577	24.28	378.7	189.4	0.5001
60MFG10	600	230	370	0.5	1347	22.44	443.5	240.6	0.5425

Table H.4 Triaxial Frequency Sweep Test results at 20°C and Deviatoric Stress of 370 kPa for Mix Type 70 with 38 Percent Manufactured Fines

Sample Name	Vertical Traction σ_1 (kPa)	Radial Traction σ_3 (kPa)	Deviatoric Stress σ_D (kPa)	Frequency (Hz)	Dynamic Modulus E_d (MPa)	Phase Angle δ (°)	RAMS	RRMS	Poisson's Ratio, ν
T70G01	600	230	370	10	2446	19.11	238.4	62.9	0.2638
T70G01	600	230	370	5	2142	19.46	274.1	75.1	0.2740
T70G01	600	230	370	1	1533	19.62	390.3	151.7	0.3887
T70G01	600	230	370	0.5	1323	18.87	453	173.8	0.3837
T70G02	600	230	370	10	2201	20.31	265.2	86.2	0.3250
T70G02	600	230	370	5	1841	20.17	319.1	111.2	0.3485
T70G02	600	230	370	1	1300	19.2	459.4	196	0.4266
T70G02	600	230	370	0.5	1154	18.19	518.1	226.4	0.4370
T70G03	600	230	370	10	2165	20.7	269.2	84.6	0.3143
T70G03	600	230	370	5	1861	20.27	315.8	108.8	0.3445
T70G03	600	230	370	1	1318	20.42	453.1	189.8	0.4189
T70G03	600	230	370	0.5	1157	18.3	517.3	216.4	0.4183
T70G04	600	230	370	10	2195	21.85	264.9	80.9	0.3054
T70G04	600	230	370	5	1894	21.46	310.3	114.4	0.3687
T70G04	600	230	370	1	1290	21.09	463.1	205.7	0.4442
T70G04	600	230	370	0.5	1111	20.05	539	237.9	0.4414
T70G05	600	230	370	10	2267	21.04	257.5	74.7	0.2901
T70G05	600	230	370	5	1964	21	299.7	89.7	0.2993
T70G05	600	230	370	1	1337	20.95	448.4	183	0.4081
T70G05	600	230	370	0.5	1149	20.49	519.7	211.3	0.4066
T70G06	600	230	370	10	2236	20.9	260.6	72	0.2763
T70G06	600	230	370	5	1950	21.26	301.5	97.6	0.3237
T70G06	600	230	370	1	1340	20.48	445.5	175.1	0.3930
T70G06	600	230	370	0.5	1159	19.41	516.3	213.3	0.4131
T70G07	600	230	370	10	3148	25.77	184.9	75.5	0.4083
T70G07	600	230	370	5	2592	25.3	226.5	100	0.4415
T70G07	600	230	370	1	1695	22.47	352.8	182.9	0.5184
T70G07	600	230	370	0.5	1432	20.65	417.8	218.7	0.5235
T70G08	600	230	370	10	2197	21.03	266.2	83.2	0.3125
T70G08	600	230	370	5	1895	20.58	310.4	102.8	0.3312
T70G08	600	230	370	1	1354	19.55	441.1	178.3	0.4042
T70G08	600	230	370	0.5	1198	18.25	499.9	207.2	0.4145
T70G09	600	230	370	10	2156	21.16	270.8	93.6	0.3456
T70G09	600	230	370	5	1877	20.27	312.7	115.8	0.3703
T70G09	600	230	370	1	1327	19.12	450.3	190.9	0.4239
T70G09	600	230	370	0.5	1181	17.88	506.7	229.5	0.4529
T70G10	600	230	370	10	2156	21.3	270.2	102.6	0.3797
T70G10	600	230	370	5	1834	20.51	319.3	120.5	0.3774
T70G10	600	230	370	1	1287	19.53	464.2	197	0.4244
T70G10	600	230	370	0.5	1139	18.23	524.9	240.6	0.4584

Table H.5 Triaxial Frequency Sweep Test results at 20°C and Deviatoric Stress of 425 kPa for Mix Type 72 with 20 Percent Manufactured Fines

Sample Name	Vertical Traction σ_1 (kPa)	Radial Traction σ_3 (kPa)	Deviatoric Stress σ_D (kPa)	Frequency (Hz)	Dynamic Modulus E_d (MPa)	Phase Angle δ (°)	RAMS	RRMS	Poisson's Ratio, ν
20MFG01	600	175	425	10	2055	20.51	282.3	78.1	0.2767
20MFG01	600	175	425	5	1805	20.43	324.2	103.6	0.3196
20MFG01	600	175	425	1	1261	20.78	471.7	186	0.3943
20MFG01	600	175	425	0.5	1092	19.85	545.1	230.1	0.4221
20MFG02	600	175	425	10	1915	22.36	302.7	95.8	0.3165
20MFG02	600	175	425	5	1640	22.14	356.9	116.8	0.3273
20MFG02	600	175	425	1	1133	21.44	525.6	216.1	0.4111
20MFG02	600	175	425	0.5	983	20.09	607.7	267.7	0.4405
20MFG04	600	175	425	10	1950	21.81	297.8	95.7	0.3214
20MFG04	600	175	425	5	1683	22.16	347	122.6	0.3533
20MFG04	600	175	425	1	1174	21.16	506.5	223	0.4403
20MFG04	600	175	425	0.5	1015	19.89	588	269.9	0.4590
20MFG05	600	175	425	10	2000	21.36	290.7	93.9	0.3230
20MFG05	600	175	425	5	1694	21.23	345.2	119.2	0.3453
20MFG05	600	175	425	1	1170	20.88	509.3	212.5	0.4172
20MFG05	600	175	425	0.5	1010	19.81	591.1	262	0.4432
20MFG06	600	175	425	10	1938	21.04	299.6	91.2	0.3044
20MFG06	600	175	425	5	1661	21.21	352	123.4	0.3506
20MFG06	600	175	425	1	1154	20.65	515.8	216.4	0.4195
20MFG06	600	175	425	0.5	1004	19.72	594	266.2	0.4481
20MFG07	600	175	425	10	1995	21.1	290.9	85.8	0.2949
20MFG07	600	175	425	5	1703	20.88	344	110.5	0.3212
20MFG07	600	175	425	1	1183	21.2	504	201.6	0.4000
20MFG07	600	175	425	0.5	1017	20.02	585.7	247.4	0.4224
20MFG08	600	175	425	10	1922	22.08	302.2	105.5	0.3491
20MFG08	600	175	425	5	1647	22.97	354.9	127.9	0.3604
20MFG08	600	175	425	1	1149	21.02	519.1	232.4	0.4477
20MFG08	600	175	425	0.5	1000	19.77	596.8	278.2	0.4662
20MFG09	600	175	425	10	1965	21.12	295.7	93.8	0.3172
20MFG09	600	175	425	5	1693	21.45	345.2	118.9	0.3444
20MFG09	600	175	425	1	1174	20.87	507.5	215.5	0.4246
20MFG09	600	175	425	0.5	1012	20.1	589.4	262.5	0.4454
20MFG10	600	175	425	10	1915	21.77	303.1	97.1	0.3204
20MFG10	600	175	425	5	1638	21.28	357.2	124.5	0.3485
20MFG10	600	175	425	1	1146	20.81	519.7	218.9	0.4212
20MFG10	600	175	425	0.5	995	19.71	599.8	265.8	0.4431
20MFG33	600	175	425	10	1974	21.2	294.5	91.2	0.3097
20MFG33	600	175	425	5	1730	21.16	338.3	116.9	0.3456
20MFG33	600	175	425	1	1206	20.4	494.2	205	0.4148
20MFG33	600	175	425	0.5	1043	19.77	572.1	249.9	0.4368

Table H.6 Triaxial Frequency Sweep Test results at 20°C and Deviatoric Stress of 425 kPa for Mix Type 72 with 40 Percent Manufactured Fines

Sample Name	Vertical Traction σ_1 (kPa)	Radial Traction σ_3 (kPa)	Deviatoric Stress σ_D (kPa)	Frequency (Hz)	Dynamic Modulus E_d (MPa)	Phase Angle δ (°)	RAMS	RRMS	Poisson's Ratio, ν
40MFG01	600	175	425	10	2066	20.39	280.3	80.9	0.2886
40MFG01	600	175	425	5	1813	20.76	322.7	108	0.3347
40MFG01	600	175	425	1	1247	21.2	476.6	194.6	0.4083
40MFG01	600	175	425	0.5	1069	20.18	558.6	238.2	0.4264
40MFG03	600	175	425	10	1936	21.62	300.1	89.2	0.2972
40MFG03	600	175	425	5	1684	21.48	347.4	121.2	0.3489
40MFG03	600	175	425	1	1157	21.15	515.1	218	0.4232
40MFG03	600	175	425	0.5	1000	20.22	595.8	267.1	0.4483
40MFG04	600	175	425	10	2226	19.9	261.6	81.2	0.3104
40MFG04	600	175	425	5	1954	20.14	300.7	112.1	0.3728
40MFG04	600	175	425	1	1376	19.52	435.2	190.2	0.4370
40MFG04	600	175	425	0.5	1190	18.63	503.3	231.9	0.4608
40MFG05	600	175	425	10	1880	21.23	308.3	93	0.3017
40MFG05	600	175	425	5	1650	21.79	354.8	121	0.3410
40MFG05	600	175	425	1	1133	21.27	525	220.1	0.4192
40MFG05	600	175	425	0.5	980	19.93	607.9	271.2	0.4461
40MFG06	600	175	425	10	2026	20.84	286.1	88.9	0.3107
40MFG06	600	175	425	5	1754	20.96	333.7	120.9	0.3623
40MFG06	600	175	425	1	1194	21.38	498.9	212.3	0.4255
40MFG06	600	175	425	0.5	1030	20.56	579.9	266.3	0.4592
40MFG07	600	175	425	10	1912	21.52	304.2	93.1	0.3060
40MFG07	600	175	425	5	1677	21.17	349.5	130	0.3720
40MFG07	600	175	425	1	1141	21.28	522.7	225.8	0.4320
40MFG07	600	175	425	0.5	989	20.23	603.2	275.7	0.4571
40MFG08	600	175	425	10	1961	21.83	296.4	101.2	0.3414
40MFG08	600	175	425	5	1678	22.03	348.9	129.8	0.3720
40MFG08	600	175	425	1	1133	21.96	526.4	238.8	0.4536
40MFG08	600	175	425	0.5	969	21.11	615.5	293.1	0.4762
40MFG09	600	175	425	10	1916	21.74	302.5	91.1	0.3012
40MFG09	600	175	425	5	1672	21.67	350	120.4	0.3440
40MFG09	600	175	425	1	1145	21.01	520.2	213.4	0.4102
40MFG09	600	175	425	0.5	991	19.89	602	259.7	0.4314
40MFG10	600	175	425	10	1906	22.06	305.3	102.5	0.3357
40MFG10	600	175	425	5	1591	22.34	368.4	127.6	0.3464
40MFG10	600	175	425	1	1100	21.35	542.7	228.4	0.4209
40MFG10	600	175	425	0.5	949	20.06	629.6	276.6	0.4393
40MFG15	600	175	425	10	2041	21.21	284	83.7	0.2947
40MFG15	600	175	425	5	1742	21.9	336.2	109.1	0.3245
40MFG15	600	175	425	1	1204	21.54	495.3	202.4	0.4086
40MFG15	600	175	425	0.5	1031	20.52	579.4	251.8	0.4346

Table H.7 Triaxial Frequency Sweep Test results at 20°C and Deviatoric Stress of 425 kPa for Mix Type 72 with 60 Percent Manufactured Fines

Sample Name	Vertical Traction σ_1 (kPa)	Radial Traction σ_3 (kPa)	Deviatoric Stress σ_D (kPa)	Frequency (Hz)	Dynamic Modulus E_d (MPa)	Phase Angle δ (°)	RAMS	RRMS	Poisson's Ratio, ν
60MFG01	600	175	425	10	3073	28.33	188.4	72.2	0.3832
60MFG01	600	175	425	5	2530	28.08	231.1	97.8	0.4232
60MFG01	600	175	425	1	1562	26.07	381.7	184.9	0.4844
60MFG01	600	175	425	0.5	1291	24.4	462.2	232.2	0.5024
60MFG02	600	175	425	10	2597	28.97	223.4	85.2	0.3814
60MFG02	600	175	425	5	2130	27.71	274.1	117.8	0.4298
60MFG02	600	175	425	1	1332	25.21	446.6	212.7	0.4763
60MFG02	600	175	425	0.5	1119	23.44	532.2	261.4	0.4912
60MFG03	600	175	425	10	2830	30.02	204.5	82.4	0.4029
60MFG03	600	175	425	5	2251	28.91	259.6	110.4	0.4253
60MFG03	600	175	425	1	1388	26.03	429.5	201.9	0.4701
60MFG03	600	175	425	0.5	1153	24.13	518.1	249.6	0.4818
60MFG04	600	175	425	10	2911	29.55	199.4	88	0.4413
60MFG04	600	175	425	5	2331	29.69	250.6	111.3	0.4441
60MFG04	600	175	425	1	1389	27.25	428.7	227.9	0.5316
60MFG04	600	175	425	0.5	1142	25.27	521.9	289.4	0.5545
60MFG05	600	175	425	10	3617	24.98	160	55.9	0.3494
60MFG05	600	175	425	5	3027	25.2	192.7	73.7	0.3825
60MFG05	600	175	425	1	1912	25.52	311.8	142.3	0.4564
60MFG05	600	175	425	0.5	1571	24.88	379.8	175.2	0.4613
60MFG06	600	175	425	10	3879	25.08	149.4	51.6	0.3454
60MFG06	600	175	425	5	3222	25.66	181.3	63.6	0.3508
60MFG06	600	175	425	1	2053	25.33	290.2	119.1	0.4104
60MFG06	600	175	425	0.5	1665	24.61	358.4	145.8	0.4068
60MFG07	600	175	425	10	2153	18.01	269.2	66.5	0.2470
60MFG07	600	175	425	5	1917	18.87	305.4	82.3	0.2695
60MFG07	600	175	425	1	1386	20.19	430	152.1	0.3537
60MFG07	600	175	425	0.5	1188	19.87	502	185.4	0.3693
60MFG08	600	175	425	10	2873	29.89	202	80.2	0.3970
60MFG08	600	175	425	5	2309	28.98	253.1	109.2	0.4315
60MFG08	600	175	425	1	1394	26.55	427.2	199.8	0.4677
60MFG08	600	175	425	0.5	1168	24.56	511	255.9	0.5008
60MFG09	600	175	425	10	2983	28.94	194.5	77	0.3959
60MFG09	600	175	425	5	2420	29.31	241.4	109.1	0.4519
60MFG09	600	175	425	1	1473	26.42	404.2	203.1	0.5025
60MFG09	600	175	425	0.5	1217	24.7	490.1	256.3	0.5230
60MFG10	600	175	425	10	2750	29.83	210.9	87.9	0.4168
60MFG10	600	175	425	5	2259	29.7	259	126.2	0.4873
60MFG10	600	175	425	1	1373	25.99	433.7	214.6	0.4948
60MFG10	600	175	425	0.5	1139	24.11	522.4	267.1	0.5113

Table H.8 Triaxial Frequency Sweep Test results at 20°C and Deviatoric Stress of 425 kPa for Mix Type 70 with 38 Percent Manufactured Fines

Sample Name	Vertical Traction σ_1 (kPa)	Radial Traction σ_3 (kPa)	Deviatoric Stress σ_D (kPa)	Frequency (Hz)	Dynamic Modulus E_d (MPa)	Phase Angle δ (°)	RAMS	RRMS	Poisson's Ratio, ν
T70G01	600	175	425	10	2191	19.44	265	61.4	0.2317
T70G01	600	175	425	5	1945	19.73	300.8	87.9	0.2922
T70G01	600	175	425	1	1349	20.91	442.3	161.9	0.3660
T70G01	600	175	425	0.5	1160	20.05	513.8	198.9	0.3871
T70G02	600	175	425	10	1972	21.26	294.8	85.9	0.2914
T70G02	600	175	425	5	1671	21.37	349.9	114.1	0.3261
T70G02	600	175	425	1	1144	20.91	520.7	216.4	0.4156
T70G02	600	175	425	0.5	993	20.15	599.5	269.2	0.4490
T70G03	600	175	425	10	1979	21.66	293.4	79.9	0.2723
T70G03	600	175	425	5	1713	21.32	341.5	112.33	0.3289
T70G03	600	175	425	1	1168	21.31	510.5	206.9	0.4053
T70G03	600	175	425	0.5	996	20.09	598.9	248.2	0.4144
T70G04	600	175	425	10	1974	21.36	294	94.7	0.3221
T70G04	600	175	425	5	1704	21.48	343.1	129.7	0.3780
T70G04	600	175	425	1	1140	22.32	522.8	238.3	0.4558
T70G04	600	175	425	0.5	966	21.29	617.1	291.3	0.4720
T70G05	600	175	425	10	2052	21.33	282.6	81.2	0.2873
T70G05	600	175	425	5	1760	21.94	332.9	106.4	0.3196
T70G05	600	175	425	1	1189	22.28	502.2	205.8	0.4098
T70G05	600	175	425	0.5	1002	21.01	595.7	254.4	0.4271
T70G06	600	175	425	10	2047	21.76	283.1	78.5	0.2773
T70G06	600	175	425	5	1767	22.37	330.8	105.1	0.3177
T70G06	600	175	425	1	1190	21.98	501	202.8	0.4048
T70G06	600	175	425	0.5	1016	20.93	587.2	251.9	0.4290
T70G07	600	175	425	10	2693	27.3	215.2	81	0.3764
T70G07	600	175	425	5	2183	26.93	267.6	108.4	0.4051
T70G07	600	175	425	1	1390	24.86	428.6	215.7	0.5033
T70G07	600	175	425	0.5	1160	23.09	514.2	268.9	0.5229
T70G08	600	175	425	10	2026	21.92	287.3	93.6	0.3258
T70G08	600	175	425	5	1738	21.66	336.5	119.2	0.3542
T70G08	600	175	425	1	1185	21.22	503	208.9	0.4153
T70G08	600	175	425	0.5	1030	20.18	578.5	256.5	0.4434
T70G09	600	175	425	10	1948	21.7	297.2	101.5	0.3415
T70G09	600	175	425	5	1696	22.14	344.5	126.9	0.3684
T70G09	600	175	425	1	1150	21.04	517.6	228.5	0.4415
T70G09	600	175	425	0.5	1001	19.98	596.2	279.2	0.4683
T70G10	600	175	425	10	1972	22.56	294	112	0.3810
T70G10	600	175	425	5	1662	22.35	351.6	137.5	0.3911
T70G10	600	175	425	1	1124	21.56	529.8	242.3	0.4573
T70G10	600	175	425	0.5	976	20.67	611.4	305.6	0.4998

Table H.9 Triaxial Frequency Sweep Test results at 20°C and Deviatoric Stress of 500 kPa for Mix Type 72 with 20 Percent Manufactured Fines

Sample Name	Vertical Traction σ_1 (kPa)	Radial Traction σ_3 (kPa)	Deviatoric Stress σ_D (kPa)	Frequency (Hz)	Dynamic Modulus E_d (MPa)	Phase Angle δ (°)	RAMS	RRMS	Poisson's Ratio, ν
20MFG01	600	100	500	10	1928	20.02	300.4	80.5	0.2680
20MFG01	600	100	500	5	1687	20.26	346.4	108.8	0.3141
20MFG01	600	100	500	1	1167	20.67	510.4	201.9	0.3956
20MFG01	600	100	500	0.5	992	19.94	600.6	255.2	0.4249
20MFG02	600	100	500	10	1798	21.97	322.4	91.2	0.2829
20MFG02	600	100	500	5	1543	22.3	378.4	124.5	0.3290
20MFG02	600	100	500	1	1039	21.38	572.6	234.3	0.4092
20MFG02	600	100	500	0.5	886	20.11	672.9	298.4	0.4435
20MFG04	600	100	500	10	1846	21.74	314.3	92.4	0.2940
20MFG04	600	100	500	5	1586	21.99	368.6	130.2	0.3532
20MFG04	600	100	500	1	1064	21.23	559.5	239.4	0.4279
20MFG04	600	100	500	0.5	910	19.97	655.6	305.6	0.4661
20MFG05	600	100	500	10	1826	21.13	318.2	97.9	0.3077
20MFG05	600	100	500	5	1565	21.34	373.6	118.5	0.3172
20MFG05	600	100	500	1	1064	20.91	560.8	236.9	0.4224
20MFG05	600	100	500	0.5	907	19.66	657.8	291.1	0.4425
20MFG06	600	100	500	10	1839	21.24	315.6	91.8	0.2909
20MFG06	600	100	500	5	1596	21.27	366	120.9	0.3303
20MFG06	600	100	500	1	1075	21.14	554.7	234.4	0.4226
20MFG06	600	100	500	0.5	920	19.89	648.5	292.7	0.4513
20MFG07	600	100	500	10	1849	20.38	313.4	90.8	0.2897
20MFG07	600	100	500	5	1590	21.26	367.6	112.8	0.3069
20MFG07	600	100	500	1	1082	21.71	551.8	220.1	0.3989
20MFG07	600	100	500	0.5	915	20.51	652.2	278.9	0.4276
20MFG08	600	100	500	10	1750	21.59	331.9	111.7	0.3365
20MFG08	600	100	500	5	1511	21.68	386.4	146.3	0.3786
20MFG08	600	100	500	1	1036	20.52	576.1	257.1	0.4463
20MFG08	600	100	500	0.5	885	19.52	674.8	315.3	0.4672
20MFG09	600	100	500	10	1822	21.51	318.1	96	0.3018
20MFG09	600	100	500	5	1575	21.36	370.7	132.1	0.3564
20MFG09	600	100	500	1	1061	21.19	562.5	239.6	0.4260
20MFG09	600	100	500	0.5	904	19.95	659.4	297.9	0.4518
20MFG10	600	100	500	10	1798	21.82	322.2	91.2	0.2831
20MFG10	600	100	500	5	1543	21.95	378.4	126.3	0.3338
20MFG10	600	100	500	1	1040	20.96	572.3	236	0.4124
20MFG10	600	100	500	0.5	891	19.63	670.2	300.8	0.4488
20MFG33	600	100	500	10	1863	21.09	311.5	85.1	0.2732
20MFG33	600	100	500	5	1608	20.99	363.6	116.4	0.3201
20MFG33	600	100	500	1	1094	20.76	545.4	221.8	0.4067
20MFG33	600	100	500	0.5	934	19.76	639.2	282.9	0.4426

Table H.10 Triaxial Frequency Sweep Test results at 20°C and Deviatoric Stress of 500 kPa for Mix Type 72 with 40 Percent Manufactured Fines

Sample Name	Vertical Traction σ_1 (kPa)	Radial Traction σ_3 (kPa)	Deviatoric Stress σ_D (kPa)	Frequency (Hz)	Dynamic Modulus E_d (MPa)	Phase Angle δ (°)	RAMS	RRMS	Poisson's Ratio, ν
40MFG01	600	100	500	10	1920	19.95	301.5	91.2	0.3025
40MFG01	600	100	500	5	1682	20.75	347.1	115.1	0.3316
40MFG01	600	100	500	1	1145	21.37	520.3	215.7	0.4146
40MFG01	600	100	500	0.5	970	20.37	615.5	268.8	0.4367
40MFG03	600	100	500	10	1859	21.24	311.7	89.1	0.2859
40MFG03	600	100	500	5	1623	21.09	360.4	126.6	0.3513
40MFG03	600	100	500	1	1079	21.92	552.4	237.8	0.4305
40MFG03	600	100	500	0.5	912	20.88	654.1	302.5	0.4625
40MFG04	600	100	500	10	1859	21.24	311.7	89.1	0.2859
40MFG04	600	100	500	5	1623	21.09	360.4	126.6	0.3513
40MFG04	600	100	500	1	1079	21.92	552.4	237.8	0.4305
40MFG04	600	100	500	0.5	912	20.88	654.1	302.5	0.4625
40MFG05	600	100	500	10	1783	21.36	325	97.4	0.2997
40MFG05	600	100	500	5	1544	21.8	378.6	128.3	0.3389
40MFG05	600	100	500	1	1041	21.2	573.7	240.9	0.4199
40MFG05	600	100	500	0.5	892	19.9	668.9	303.8	0.4542
40MFG06	600	100	500	10	1886	20.68	308	95	0.3084
40MFG06	600	100	500	5	1639	21.32	356.8	125.4	0.3515
40MFG06	600	100	500	1	1096	21.36	544.5	223.8	0.4110
40MFG06	600	100	500	0.5	931	20.29	640.9	283.8	0.4428
40MFG07	600	100	500	10	1809	21.2	321.3	102.1	0.3178
40MFG07	600	100	500	5	1550	21.95	377.4	127.2	0.3370
40MFG07	600	100	500	1	1044	21.25	572.2	250.8	0.4383
40MFG07	600	100	500	0.5	884	19.93	674.3	308.4	0.4574
40MFG08	600	100	500	10	1848	21.09	314.2	108.5	0.3453
40MFG08	600	100	500	5	1572	21.75	371.4	141.4	0.3807
40MFG08	600	100	500	1	1055	21.34	565.9	250.6	0.4428
40MFG08	600	100	500	0.5	894	20.26	665.4	309.3	0.4648
40MFG09	600	100	500	10	1760	21.6	329	96.2	0.2924
40MFG09	600	100	500	5	1520	21.76	384.4	124	0.3226
40MFG09	600	100	500	1	1030	21.07	578.7	234	0.4044
40MFG09	600	100	500	0.5	881	19.95	677.3	290.6	0.4291
40MFG10	600	100	500	10	1771	21.93	327.8	103.5	0.3157
40MFG10	600	100	500	5	1492	22.22	391.3	133.9	0.3422
40MFG10	600	100	500	1	1010	21.09	590.9	241.3	0.4084
40MFG10	600	100	500	0.5	863	19.87	690.7	294.6	0.4265
40MFG15	600	100	500	10	1811	20.75	319.9	71	0.2219
40MFG15	600	100	500	5	1595	21.65	365.8	99.5	0.2720
40MFG15	600	100	500	1	1083	21.39	550.8	207.3	0.3764
40MFG15	600	100	500	0.5	932	19.94	640.5	265.3	0.4142

Table H.11 Triaxial Frequency Sweep Test results at 20°C and Deviatoric Stress of 500 kPa for Mix Type 72 with 60 Percent Manufactured Fines

Sample Name	Vertical Traction σ_1 (kPa)	Radial Traction σ_3 (kPa)	Deviatoric Stress σ_D (kPa)	Frequency (Hz)	Dynamic Modulus E_d (MPa)	Phase Angle δ (°)	RAMS	RRMS	Poisson's Ratio, ν
60MFG01	600	100	500	10	2880	27.98	200.9	75.5	0.3758
60MFG01	600	100	500	5	2370	27.7	246.5	102.3	0.4150
60MFG01	600	100	500	1	1435	25.89	415.9	200.9	0.4830
60MFG01	600	100	500	0.5	1172	24.14	508.7	258.8	0.5087
60MFG02	600	100	500	10	2429	28.43	238.2	92.2	0.3871
60MFG02	600	100	500	5	1975	27.1	295.7	122.7	0.4149
60MFG02	600	100	500	1	1217	24.68	490.6	229.2	0.4672
60MFG02	600	100	500	0.5	1016	22.91	586.8	287.6	0.4901
60MFG03	600	100	500	10	2608	29.05	222.5	88.9	0.3996
60MFG03	600	100	500	5	2106	29.07	277	115.5	0.4170
60MFG03	600	100	500	1	1272	25.56	470.1	220.8	0.4697
60MFG03	600	100	500	0.5	1045	23.75	570.1	268.3	0.4706
60MFG04	600	100	500	10	2702	28.88	214.9	85.3	0.3969
60MFG04	600	100	500	5	2167	29.14	269.4	112.5	0.4176
60MFG04	600	100	500	1	1296	26.38	460.3	222.4	0.4832
60MFG04	600	100	500	0.5	1061	24.24	562.4	301.7	0.5365
60MFG05	600	100	500	10	3361	24.79	172.5	58.2	0.3374
60MFG05	600	100	500	5	2828	25.12	206.2	80.5	0.3904
60MFG05	600	100	500	1	1761	25.23	338.5	152	0.4490
60MFG05	600	100	500	0.5	1440	24.5	414.2	197	0.4756
60MFG06	600	100	500	10	3755	24.46	154.3	49.1	0.3182
60MFG06	600	100	500	5	3113	25.15	187.6	64.7	0.3449
60MFG06	600	100	500	1	1922	25.79	310.7	124.2	0.3997
60MFG06	600	100	500	0.5	1544	24.96	386.2	158.4	0.4102
60MFG07	600	100	500	10	1961	17.43	296.1	58.2	0.1966
60MFG07	600	100	500	5	1740	18.18	335.5	78.1	0.2328
60MFG07	600	100	500	1	1256	19.9	474.4	160.2	0.3377
60MFG07	600	100	500	0.5	1073	19.62	555.7	205.7	0.3702
60MFG08	600	100	500	10	2744	29.05	211.4	84	0.3974
60MFG08	600	100	500	5	2173	28.25	268.1	113.1	0.4219
60MFG08	600	100	500	1	1290	25.74	462.9	215.8	0.4662
60MFG08	600	100	500	0.5	1064	23.71	560.6	273	0.4870
60MFG09	600	100	500	10	2783	28.56	208.6	79.2	0.3797
60MFG09	600	100	500	5	2255	27.92	258.9	106.9	0.4129
60MFG09	600	100	500	1	1366	26.01	436.3	202.4	0.4639
60MFG09	600	100	500	0.5	1130	24.16	527	261.3	0.4958
60MFG10	600	100	500	10	2618	28.79	221.4	89.7	0.4051
60MFG10	600	100	500	5	2089	28.25	279.4	118.5	0.4241
60MFG10	600	100	500	1	1266	25.45	470.1	226.7	0.4822
60MFG10	600	100	500	0.5	1047	23.82	569.4	284.7	0.5000

Table H.12 Triaxial Frequency Sweep Test results at 20°C and Deviatoric Stress of 500 kPa for Mix Type 70 with 38 Percent Manufactured Fines

Sample Name	Vertical Traction σ_1 (kPa)	Radial Traction σ_3 (kPa)	Deviatoric Stress σ_D (kPa)	Frequency (Hz)	Dynamic Modulus E_d (MPa)	Phase Angle δ (°)	RAMS	RRMS	Poisson's Ratio, ν
T70G01	600	100	500	10	2066	19.22	281	60.5	0.2153
T70G01	600	100	500	5	1846	19.2	316.1	92.5	0.2926
T70G01	600	100	500	1	1267	20.75	471	177.7	0.3773
T70G01	600	100	500	0.5	1071	20.02	556.4	212.2	0.3814
T70G02	600	100	500	10	1815	20.33	320	86.9	0.2716
T70G02	600	100	500	5	1555	21.03	375.7	112.2	0.2986
T70G02	600	100	500	1	1067	20.44	559.3	227.8	0.4073
T70G02	600	100	500	0.5	916	19.44	651.2	287.6	0.4416
T70G03	600	100	500	10	1851	20.82	313.4	82.5	0.2632
T70G03	600	100	500	5	1592	21.1	366.9	114.5	0.3121
T70G03	600	100	500	1	1085	20.7	549.8	221.4	0.4027
T70G03	600	100	500	0.5	931	19.71	641.5	287.6	0.4483
T70G04	600	100	500	10	1877	20.57	309.4	90.1	0.2912
T70G04	600	100	500	5	1609	22.11	363.1	120.3	0.3313
T70G04	600	100	500	1	1089	21.43	548.3	232.8	0.4246
T70G04	600	100	500	0.5	931	20.14	641.2	305.8	0.4769
T70G05	600	100	500	10	1921	20.78	302.1	74.3	0.2459
T70G05	600	100	500	5	1667	20.85	350	99.9	0.2854
T70G05	600	100	500	1	1121	21.52	533	208.4	0.3910
T70G05	600	100	500	0.5	950	20.34	627.3	263.4	0.4199
T70G06	600	100	500	10	1931	20.92	299.8	71.9	0.2398
T70G06	600	100	500	5	1669	21.64	349.6	96.8	0.2769
T70G06	600	100	500	1	1123	21.41	532	194.8	0.3662
T70G06	600	100	500	0.5	952	20.4	626.5	254.6	0.4064
T70G07	600	100	500	10	2478	26.51	233.9	82.7	0.3536
T70G07	600	100	500	5	2069	26.31	282.2	116.1	0.4114
T70G07	600	100	500	1	1293	23.87	462	217.4	0.4706
T70G07	600	100	500	0.5	1072	22.2	555.2	274.8	0.4950
T70G08	600	100	500	10	1875	20.88	310	96.4	0.3110
T70G08	600	100	500	5	1628	21.13	359.1	129.2	0.3598
T70G08	600	100	500	1	1093	20.68	546.5	226.7	0.4148
T70G08	600	100	500	0.5	935	19.65	638.4	285.1	0.4466
T70G09	600	100	500	10	1831	21.55	316.4	109.8	0.3470
T70G09	600	100	500	5	1563	22.2	373.8	137.5	0.3678
T70G09	600	100	500	1	1052	21.17	567.2	254.9	0.4494
T70G09	600	100	500	0.5	905	19.83	659.5	312.8	0.4743
T70G10	600	100	500	10	1858	21.25	312.4	101.8	0.3259
T70G10	600	100	500	5	1587	21.58	368.5	133.7	0.3628
T70G10	600	100	500	1	1066	20.78	560.7	246.6	0.4398
T70G10	600	100	500	0.5	912	19.9	653.8	303	0.4634

APPENDIX I. SAMPLE SIZE ANALYSIS BASED ON MECHANISTIC CHARACTERIZATION OF THE RESEARCH MIXES

Table I.1 Relationship of Sample Size and Level of Confidence for Dynamic Modulus at 10 Hz and Deviatoric Stress of 500 kPa across Research Mixes at a Margin of Error of 200 MPa

Margin of Error = 200 MPa

Level of Confidence	Z	Sample Size (n)			
		T72(20%MF)	T72(40%MF)	T72(60%MF)	T70(38%MF)
99%	2.58	1	1	40	7
95%	1.96	1	1	23	4
90%	1.65	1	1	16	3
75%	1.15	1	1	8	1
50%	0.68	1	1	3	1

Table I.2 Relationship of Sample Size and Level of Confidence for Recoverable Axial Microstrain at 10 Hz and Deviatoric Stress of 500 kPa across Research Mixes at a Margin of Error of 20×10^{-6}

Margin of Error = 20 (10^{-6})

Level of Confidence	Z	Sample Size (n)			
		T72(20%MF)	T72(40%MF)	T72(60%MF)	T70(38%MF)
99%	2.58	1	1	24	11
95%	1.96	1	1	14	6
90%	1.65	1	1	10	4
75%	1.15	1	1	5	2
50%	0.68	1	1	2	1

Table I.3 Relationship of Sample Size and Level of Confidence for Recoverable Radial Microstrain at 10 Hz and Deviatoric Stress of 500 kPa across Research Mixes at a Margin of Error of 10×10^{-6}

Margin of Error = 10 (10^{-6})

Level of Confidence	Z	Sample Size (n)			
		T72(20%MF)	T72(40%MF)	T72(60%MF)	T70(38%MF)
99%	2.58	5	7	16	14
95%	1.96	3	4	9	8
90%	1.65	2	3	6	6
75%	1.15	1	1	3	3
50%	0.68	1	1	1	1

Table I.4 Relationship of Sample Size and Level of Confidence for Poisson's Ratio at 10 Hz and Deviatoric Stress of 500 kPa across Research Mixes at a Margin of Error of 0.03

Margin of Error = 0.03

Level of Confidence	Z	Sample Size (n)			
		T72(20%MF)	T72(40%MF)	T72(60%MF)	T70(38%MF)
99%	2.58	3	8	30	16
95%	1.96	2	4	17	9
90%	1.65	1	3	12	7
75%	1.15	1	1	6	3
50%	0.68	1	1	2	1

Table I.5 Relationship of Sample Size and Level of Confidence for Phase Angle at 10 Hz and Deviatoric Stress of 500 kPa across Research Mixes at a Margin of Error of 2.0 Degrees

Margin of Error = 2.0 degrees

Level of Confidence	Z	Sample Size (n)			
		T72(20%MF)	T72(40%MF)	T72(60%MF)	T70(38%MF)
99%	2.58	1	1	23	6
95%	1.96	1	1	13	4
90%	1.65	1	1	9	3
75%	1.15	1	1	5	1
50%	0.68	1	1	2	1

APPENDIX J. LIFE CYCLE COST ANALYSIS FOR SASKATCHEWAN HMAC PAVEMENTS

Life Cycle Cost Analysis for Saskatchewan HMAC Pavements

Assumptions:

- Typical SDHT pavement life cycle is 25 years.
- Initial capital construction costs and routine maintenance costs over life cycle are the same for all pavement performance scenarios.
- 500 km of the SDHT pavement network is resurfaced with asphalt concrete on an annual basis.
- Pavement performance was assessed assuming that structural design of pavement is adequate to handle field state conditions over the 25 year pavement service life.
- Following standard SDHT practice, an interest rate of 3 percent was applied in present value calculations.
- HMAC aggregate is valued at $15 per metric tonne in today's dollars.

The preservation treatment details and their respective costs that were assumed for the purposes of the economic analysis are shown in Table J. 1.

Table J. 1 Preservation Treatment Costs and Aggregate Needs

	Treatment Cost (PV $ Per Kilometre)	Aggregate Required (metric tonnes per kilometre)
Mill and Replace 60 mm both driving lanes	50,000	1,066
Strip Seal in Wheel Paths	12,000	114
Microsurfacing in both Driving Lanes	37,000	211
Full Seal in both Driving Lanes	22,000	211

Table J. 2 Life Cycle Treatment Costs for a Failed Pavement - Plastic Flow Rutting in the First 5 Years of Service Life

	Annual Treatment Cost per Km of Two Lane Highway					
Year	**Routine**	**Light**	**Medium**	**Heavy**	**Total**	**Present Value**
0						
1	$450				$450	$437
2	$464				$464	$437
3	$477				$477	$437
4	$492				$492	$437
5	$506			$56,275	$56,782	$48,981
6	$348				$348	$291
7	$358				$358	$291
8	$369				$369	$291
9	$380				$380	$291
10	$391				$391	$291
11	$269				$269	$194
12	$277				$277	$194
13	$285				$285	$194
14	$294				$294	$194
15	$303	$18,151			$18,454	$11,845
16	$234				$234	$146
17	$241				$241	$146
18	$248				$248	$146
19	$255				$255	$146
20	$263		$64,880		$65,143	$36,068
21	$271				$271	$146
22	$279				$279	$146
23	$287				$287	$146
24	$296				$296	$146
25	$305				$305	$146
Present Value	$6,068	$11,650	$35,922	$48,544		
Total Present Value over 25 Year Life Cycle						**$102,184**

Treatment Description

Light Strip seals in wheel paths (1m width each), at $3/m^2 ($12,000/km)

Medium Microsurfacing both lanes, $5/m^2 ($37,000/km)

Heavy Mill and replace 60 mm of rutted layer, both lanes ($50,000/km)

Annual Interest Rate = 3%

Table J. 3 Life Cycle Treatment Costs for a Typical SDHT Pavement - Poor Rutting in Year 15 of Service Life

	Annual Treatment Cost per Km of Two Lane Highway				
Year	**Routine**	**Light**	**Medium**	**Total**	**Present Value**
0					
1	$450			$450	$437
2	$464			$464	$437
3	$477			$477	$437
4	$492			$492	$437
5	$506			$506	$437
6	$348			$348	$291
7	$358			$358	$291
8	$369			$369	$291
9	$380			$380	$291
10	$391	$15,657		$16,049	$11,942
11	$269			$269	$194
12	$277			$277	$194
13	$285			$285	$194
14	$294			$294	$194
15	$303		$55,966	$56,268	$36,117
16	$234			$234	$146
17	$241			$241	$146
18	$248			$248	$146
19	$255			$255	$146
20	$263			$263	$146
21	$271			$271	$146
22	$279			$279	$146
23	$287			$287	$146
24	$296			$296	$146
25	$305			$305	$146
Present Value	$6,068	$11,650	$35,922		
Total Present Value over 25 Year Life Cycle					**$53,641**

Treatment Description

Light Strip seals in wheel paths, $3/m^2 ($12,000/km)

Medium Microsurfacing both lanes, $5/m^2 ($37,000/km)

Annual Interest Rate = 3%

Table J. 4 Life Cycle Treatment Costs for a Well-Performing Pavement - Rutting Remains Good over 25 Years of Service Life

Year	Annual Treatment Cost per Km of Two Lane Highway				
	Routine	Light	Medium	Total	Present Value
0					
1	$450			$450	$437
2	$464			$464	$437
3	$477			$477	$437
4	$492			$492	$437
5	$506			$506	$437
6	$348			$348	$291
7	$358			$358	$291
8	$369			$369	$291
9	$380			$380	$291
10	$391			$391	$291
11	$269			$269	$194
12	$277			$277	$194
13	$285			$285	$194
14	$294			$294	$194
15	$303	$33,277		$33,579	$21,553
16	$234			$234	$146
17	$241			$241	$146
18	$248			$248	$146
19	$255			$255	$146
20	$263			$263	$146
21	$271			$271	$146
22	$279			$279	$146
23	$287			$287	$146
24	$296			$296	$146
25	$305			$305	$146
Present Value	$6,068	$21,359			
Total Present Value over 25 Year Life Cycle					**$27,427**

Treatment Description

Light Full seal, 9 mm top size, both lanes $3/m^2 ($22,000/km)

Annual Interest Rate = 3%