

Development of High Rut-Resistant Asphalt Mixes for Municipal Pavements

by

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Abstract

Rutting is a major form of pavement distress, particularly at intersections and bus stops, where slow-moving and frequent stopping and accelerating vehicular loads accelerate surface deformation. In these locations, asphalt mixes often fail due to low stiffness. The contributing factors are rounded aggregates that weaken the aggregate skeleton, excessive fines, high binder content, and inadequate compaction. Since these high-stress areas experience traffic loads different from those in general roadway sections, standard pavement materials may not provide sufficient strength and durability. To address this challenge, the study focused on the development of high rut-resistant Hot Mix Asphalt (HMA) mixtures specifically designed for heavy-duty municipal pavements. The research emphasizes optimizing material selection, aggregate gradation, binder properties, and mix design parameters to enhance the structural integrity and durability of asphalt under high traffic loads.

As part of this study, aggregates from multiple sources were assessed based on angularity, abrasion resistance, and water absorption, with highly angular aggregates selected to enhance the internal structure of the mix. A 12.5 mm Nominal Maximum Aggregate Size (NMAS) laboratory mix was designed using the selected aggregates and a 64-34P asphalt binder, with the Bailey method applied to optimize aggregate packing and improve stability. In addition to the laboratory-designed mix, five plant-produced mixes were collected from three City of Winnipeg pilot projects: two 12.5 mm NMAS Superpave surface course (SP1) mixes and three 19 mm NMAS binder course (SP2) mixes. Together, these six mixes formed the test matrix for performance evaluation. Each mix was tested for rutting resistance using the Hamburg Wheel Tracking Test (HWTT), moisture sensitivity through the Tensile Strength Ratio (TSR), and stiffness performance using the Dynamic Modulus Test (DMT).

Results showed that the laboratory-produced SP1 and Project 3 mixes demonstrated high rutting resistance. They satisfied the minimum TSR requirement of 80%. At the same time, Project 1 mixes also showed superior rutting resistance and still met the minimum Indirect Tensile Strength (ITS) criterion, despite failing to comply with the minimum 80% TSR requirement. In contrast, Project 2 mixes exhibited poor rutting resistance, with Project 2 SP2 exceeding the 14 mm rut depth limit at 20,000 passes, a deficiency linked to lower Fine Aggregate Angularity (FAA) and inferior volumetric properties. Correlation analysis revealed that VMA and effective binder content strongly influenced rutting performance, while TSR values correlated positively with asphalt content and VMA, emphasizing the need for a balanced aggregate-binder structure. However, weak correlations between HWTT, TSR, and DMT highlight the limitations of relying solely on moisture sensitivity or stiffness tests for mix evaluation.

The findings suggest that rut-resistant mixes can be realistically achieved by optimizing volumetric properties, using angular aggregates with controlled gradation, and incorporating polymer-modified binders. Moreover, adopting the Bailey Method for gradation checks and incorporating performance-based specifications, similar to practices in municipalities such as Edmonton and Toronto, would significantly enhance Winnipeg's mix design framework. Finally, integrating low-temperature cracking tests and long-term field evaluations is recommended to ensure durability under the city's harsh freeze-thaw climate. Together, these insights provide a practical pathway toward designing more durable and rut-resistant municipal pavements.

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Dedication

To my beloved grandmothers, maternal and paternal, who passed away in Pakistan during this thesis. Though distant in miles, their love and memory remained close to my heart and gave me strength throughout this journey.

To my parents, whose unwavering support, encouragement, and sacrifices made this achievement possible.

To my sisters, whose unwavering love and support have carried me through every step of this journey

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Abbreviation

AASHTO	American Association of State Highway and Transportation Officials
APA	Asphalt Pavement Analyzer
AC	Asphalt Content
ATB	Asphalt Treated Base
COW	City of Winnipeg
DMT	Dynamic Modulus Test
DOT	Department of Transportation
$ E^* $	Dynamic Modulus
FAA	Fine Aggregate Angularity
FHWA	Federal Highway Administration
G_{mb}	Bulk Specific Gravity
G_{mm}	Maximum Specific Gravity
HMA	Hot-Mix Asphalt
HWTT	Hamburg Wheel-Tracking Test
IDT	Indirect Tension
MF	Manufactured Fines
NF	Natural Fines
NMAS	Nominal Maximum Aggregate Size
OBC	Optimum Binder Content
OGFC	Open Graded Friction Course
PGAC	Performance Graded Asphalt Cement
PG	Performance Grade
PMA	Polymer Modified Asphalt
TSR	Tensile Strength Ratio
SP	Superpave

SIP	Stripping Inflection Point
SMA	Stone Mastic Asphalt
Va	Air Voids
VFA	Voids Filled with Asphalt
VMA	Voids in Mineral Aggregates

Chapter 1. Introduction

1.1 Problem Statement

As infrastructure continues to age and urban development expands, municipalities face increasing challenges in ensuring the durability and long-term performance of their extensive road networks. Although various defects can negatively impact pavement service life, rutting is among the most critical forms of distress, often leading to premature pavement failure and poses significant safety risks, particularly due to hydroplaning. Over the past few decades, increasing traffic volumes have subjected pavements to higher stresses, which, in turn, have increased the rutting problem in Hot Mix Asphalt (HMA) pavements (Brown & Bassett, 1990; Hajj et al., 2011).

Rutting, or permanent deformation of asphalt pavement, is characterized by surface depressions forming along the wheel paths (Du et al., 2018). Traditionally, rutting was considered a problem primarily associated with the subgrade layers, as reflected in early pavement design models such as the 1986 AASHTO design guide. However, subsequent research has demonstrated that even high-quality asphalt layers are also susceptible to significant deformation (Parker & Brown, 1990). This distress is not limited to highways; it is often more pronounced at intersections where the stiffness of the asphalt mix decreases under slow-moving and stationary loads. The frequent stopping of vehicles generates elevated shear stresses, making pavements in these areas particularly vulnerable to rutting. Instability rutting, characterized by wheel path depressions within the asphalt layers, can result from several factors, including excessive binder content combined with insufficient air voids, the presence of rounded aggregates that prevent interlocking, along with a high proportion of fine dust particles (Brown & Bassett, 1990). Within the pavement structure, the asphalt layer is the most susceptible to rutting, with approximately 85% to 95% of the total permanent deformation typically occurring in this layer (Du et al., 2018).

Approaches to addressing the rutting problem vary significantly among municipalities. While some cities have adopted advanced and resource-efficient strategies, others continue to rely on traditional mix design practices. In recent years, Canada has witnessed considerable advancements in asphalt pavement technology, with major municipalities such as Toronto, Vancouver, Calgary, and Edmonton leading the way. These cities have implemented performance-based specifications and transitioned toward modern practices, including the adoption of Superpave mix design, premium aggregate blends, and polymer-modified asphalt binders. Some municipalities also use performance tests for informational and verification purposes as well but not as a part of acceptance criteria.

Winnipeg, in terms of infrastructure, is comparable to those cities, with an extensive road network with around 5.17 km of road lanes for every 1000 residents (Engage Winnipeg, 2025). However, due to ever-increasing traffic loads and climate change, the city faces challenges in maintaining the longevity of pavements and intersections in high-traffic zones. This is reflected in the City of Winnipeg's Road maintenance budget, which has been rising over the years due to the deteriorating condition of roads. City administration allocated \$169.3 million for road improvement in 2025, with a 15.8% increase over 2024 (City of Winnipeg, 2025).

As part of the current practices, most of Winnipeg's residential roads are constructed as flexible pavements, while regional road networks are initially constructed as rigid pavements, which are later transitioned into composite pavement systems during subsequent life cycles (Shalaby, 2005). However, these asphalt overlays are particularly vulnerable to rutting due to the stiffness contrast between the underlying rigid pavement and the overlaid asphalt layer. This mismatch causes stress concentrations at the pavement interface and limits the overlay's ability to deform uniformly, resulting in accelerated surface rutting (Cheng et al., 2023). These construction practices, coupled

with a continued reliance on traditional Marshall mix designs, have further intensified the rutting problem in recent years. Although Marshall-designed mixes have historically provided satisfactory performance, their limitations have become increasingly evident under modern traffic demands and the changing climatic conditions, often resulting in premature failures. Asi (2007) emphasized that the Marshall mix design method does not adequately address the shear susceptibility of asphalt mixtures, as the hammer-blow compaction method and stability/flow tests fail to replicate real-world loading conditions. In contrast, the Superpave system, developed under the Strategic Highway Research Program (SHRP), offers a performance-based approach that effectively evaluates a mixture's resistance to deformation under shear stresses, along with its performance across extreme temperature ranges, and the compatibility between asphalt binders and aggregates.



Figure 1.1 Rutting at an intersection in Winnipeg

In Winnipeg, the problem is not limited to mix design methodologies; there are some challenges posed by the available materials in achieving a solution to the rutting problem. The rounded nature of the glacial sand source that is locally available allows movement of adjacent particles instead of providing interlocking for better resistance against deformation. Manitoba's history is notable

for the collision of two ice sheets, which mixed limestone bedrock fragments from the northwest with granite from the northeast Canadian shield. As a result, sand and gravel deposits vary in composition based on location. Aggregate pits north and east of Lake Winnipeg and within 100 km of the Ontario border have over 70% granite. In contrast, the pits to the south and the west of these areas contain 50 to 80% limestone. The Geological Map of Manitoba supports this observation, showing that areas near Winnipeg predominantly consist of limestone and dolomite, rich in carbonate minerals. Ramsay et al. (1974) emphasized the role of aggregate mineralogy in determining material strength, noting that aggregates with higher carbonate content are weaker than those with silicate minerals. The weak skeletal structure of limestone and dolomite affects their abrasion resistance, leading to rounded aggregates that lack the interlocking properties needed for resistance against permanent deformation. When combined with locally available glacial sand, which also contains a significant number of rounded particles, the aggregate blend may be more economical but may fall short of meeting performance requirements for rutting resistance.

Addressing the issue of permanent deformation in Winnipeg requires a comprehensive solution that considers higher-quality aggregate sources with greater angularity and interlocking potential. Incorporating polymer-modified asphalt (PMA) binders in the mix can also enhance the performance of city pavements in resisting rutting. Asphalt concrete is a critical component of the city's road infrastructure, whether in new construction, overlays, or rehabilitation; even modest improvements in rutting resistance can have a significant long-term impact.

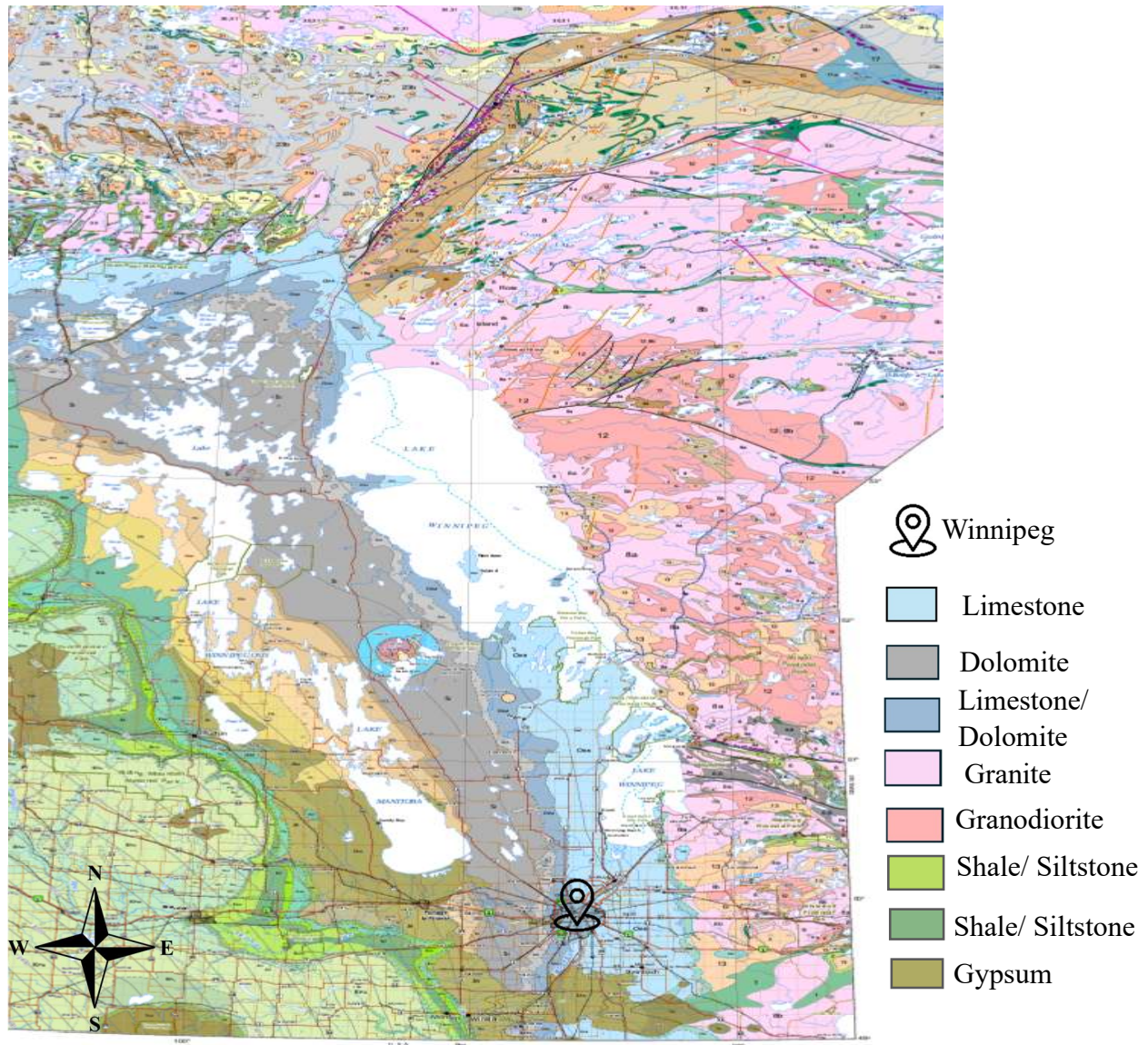


Figure 1.2 Geological Map of Manitoba (Government of Manitoba, 2024)

1.2 Purpose and Objectives

This research aimed to develop rut-resistant mixes that can sustain heavy traffic loads by increasing the stiffness of the asphalt mix. The effect of aggregate selection, binder modification, and gradation optimization was evaluated through performance testing. The overall objectives of the research are as follows:

- The objective of this study was to develop asphalt mixes for heavy-duty pavements in Winnipeg with enhanced rutting resistance by incorporating high-quality aggregates and Polymer Modified Asphalt (PMA). Heavy-duty pavements, such as intersections, bus stops, and industrial truck routes, are exposed to frequent braking, acceleration, and slow-moving heavy loads that generate high shear stresses within the asphalt layer.
- Evaluate the moisture sensitivity and rutting potential of lab-produced asphalt mix using the Moisture Sensitivity Test, Hamburg Wheel Tracking Test (HWTT), and Dynamic Modulus Test.
- Evaluate plant-produced Superpave and Marshall mixes' rutting performance based on proposed specifications. Evaluate moisture sensitivity and rutting potential using the moisture sensitivity test, Hamburg wheel tracking test, and Dynamic Modulus Test.

1.3 Research Methodology

The experimental program was conducted in three phases: Phase I, laboratory mix design and performance evaluation; Phase II, analysis of plant-produced mixes; and Phase III, forensic evaluation of plant-produced mixes to identify redundancies.

During the first testing phase, an asphalt mix was produced in the laboratory using high-quality aggregates and the Superpave mix design method, in accordance with the City of Winnipeg's proposed draft specification CW3410 R13. In contrast to current practices, aggregate sources located farther from the city were investigated for consensus properties, and aggregates with higher angularity were selected and combined with polymer-modified asphalt. The mix was then compacted to meet the volumetric property requirements as specified. Tensile Strength Ratio (TSR), Hamburg Wheel Tracking Test (HWTT), and Dynamic Modulus Test (DMT) were

conducted on the specimens to evaluate moisture sensitivity, rutting resistance, and elastic modulus, respectively.

For Phase II, five loose asphalt mixtures were sampled from different pilot projects across the city. Of these, four mixes were developed using the Superpave mix design procedure, while one was prepared using the Marshall method. The mixtures were labeled as Project 1 SP1, Project 1 SP2, Project 2 SP1, Project 2 SP2, and Project 3 MS1. The SP1 and MS1 mixtures were sourced from the surface course layer with a 12.5 mm nominal maximum aggregate size (NMAS), whereas the SP2 mixtures were taken from the binder course layer with a 19 mm NMAS. Laboratory compaction was carried out to produce test specimens from these field-produced mixes.

The performance of all five mixtures was then assessed through a series of laboratory evaluations, including the Hamburg Wheel Tracking Test (HWTT) to measure rutting resistance, the Tensile Strength Ratio (TSR) test to evaluate moisture susceptibility, and the Dynamic Modulus Test (DMT) to determine stiffness properties.

As part of Phase III, a forensic evaluation was conducted on the Project 1 and Project 2 mixtures. Two specimens from each mix were analyzed using the ignition oven procedure (AASHTO T308) to quantify the actual binder content based on mass loss at elevated temperatures. After binder removal, the recovered aggregates were tested using the Fine Aggregate Angularity (FAA) method (AASHTO T304) to compare the angularity of the plant-produced mixes with their respective design specifications.

1.4 Thesis Organization

The thesis is divided into five chapters

- Chapter 1 highlights the problem along with the background statement.

- Chapter 2 presents a detailed literature review on the comparison of municipalities on their approach to countering rutting problems. The mechanism of rutting and moisture-induced damage has also been discussed along with the factors affecting rutting performance.
- Chapter 3 includes the research methodology adopted to achieve the objectives, material properties, sample preparation and laboratory test procedures are discussed.
- Chapter 4 presents the results and discussion of all six mixes.
- In Chapter 5, the outcome of TSR, HWTT and DMT has been discussed along with the interpretation of results.
- The summary of research, along with the conclusion and recommendations, has been included in Chapter 6.

Chapter 2. Literature Review

2.1 Evaluation of rutting improvement in the context of the new specification

The City of Winnipeg specification CW 3410, Revision 12, outlines two mixes, one surface mix, Type-IA, and one binder mix, Type 3. Notably, both mixes are Marshall mixes, and no Superpave mix is specified in the current document. However, the city has recently introduced a draft of revision 13, which incorporates two Superpave mixes alongside the existing Marshall mixes. This update represents a significant improvement over the current specification.

To develop high rut-resistant mixes for heavy-duty pavements, it is essential to use higher-quality aggregates. Current mixes for the City of Winnipeg projects utilize local limestone and gravel sources, which do not consistently provide the necessary durability under heavy traffic loads. Incorporating harder aggregates such as granite or basalt can significantly enhance the mix's performance. In addition to using high-quality aggregates, incorporating Polymer Modified Binders (PMBs) in the mixtures can enhance rutting resistance and enable superior performance across a wider range of temperatures. The proposed draft version of CW 3410 R13 specifies the required consensus properties of aggregates, including fine aggregate angularity, percent abrasion, and percent fractured faces. Apart from these changes, the city has also introduced polymer-modified asphalt binders in the specifications with PG 64-34P for the SP1 mix, which is a 12.5mm NMAS surface course mix and 58-34P for SP2 base course mixes (19.5mm NMAS). These new additions in the specification would promote the selection of aggregate sources that exhibit higher consensus properties and, therefore, lead to mixes that are superior in terms of aggregate skeleton, stiffness, and consequently rutting resistance.

2.2 Comparison of City Specifications and Approaches for Rut-Resistant Asphalt Mixes

The comparison of rut-resistant mixes across various cities reveals distinct approaches:

2.2.1 City of Toronto

Superpave 12.5 FC1 and FC2 are fine-graded surface mixes designed for high-traffic urban roads, with FC1 intended for Category C traffic and FC2 for the heavier Category D and E traffic. Both mixes prioritize superior rutting resistance and skid performance through optimized aggregate selection and a higher proportion of fine material. Unlike conventional 12.5 mm NMA Superpave mixes, which are coarse-graded and designed primarily for general structural stability, FC1 and FC2 are gap-graded, meaning certain intermediate sieve sizes are deliberately reduced or omitted. This creates a coarse aggregate skeleton that provides high interlock for structural stability while allowing fines and binder to fill the voids, enhancing rutting resistance and surface friction. FC1 provides high performance for moderately heavy traffic areas, whereas FC2 is slightly coarser in the upper fractions to withstand very high loads, making it suitable for major arterial roads, intersections, and truck routes with extreme loading conditions. Toronto also includes Stone Mastic Asphalt in its specifications, though this mix is beyond the scope of this comparison.

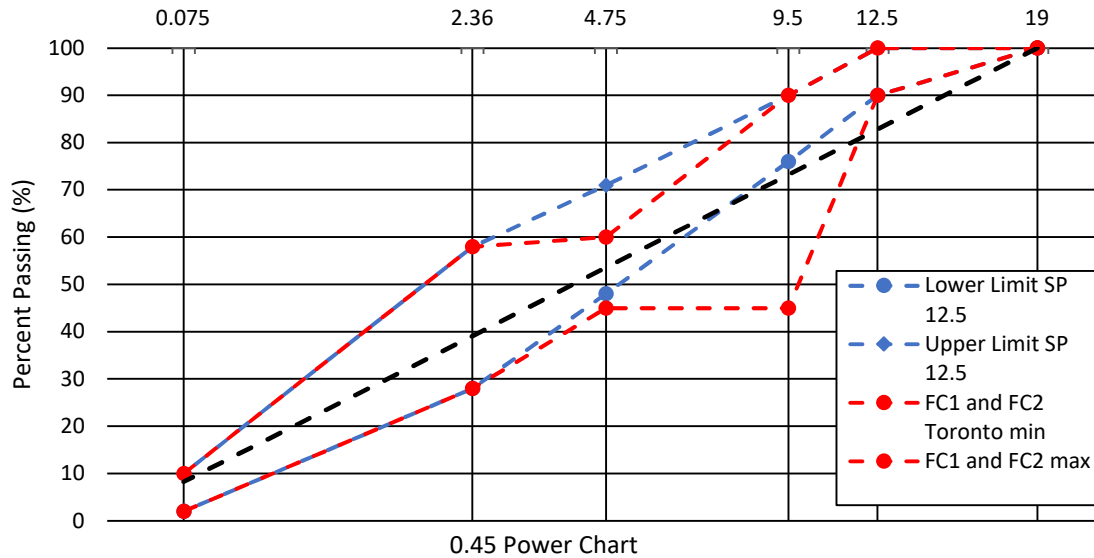


Figure 2.1 Comparison of COW SP1 mix with FC1 and FC2 mixes from the City of Toronto

A comparison of the gradation of the two mixes in Figure 2.1 shows that the FC1 mixes specified by the City of Toronto are gap-graded with a coarser coarse aggregate skeleton compared to the proposed SP1 and SP2 mixes under the City of Winnipeg's CW 3410 R13 specification. This coarser aggregate structure, combined with a higher proportion of fine material to fill voids, contributes to a stiffer mix that enhances resistance to permanent deformation and rutting. The specification also requires performance-based evaluation using tests such as the Hamburg Wheel Tracking Test (HWTT) and the Flow Number (FN) test to assess rutting potential. Mixes failing to meet the performance criteria must be redesigned, ensuring adequate binder content, aggregate interlock, and overall durability for high-stress municipal pavements.

2.2.2 City of Vancouver

City of Vancouver construction specifications specify the use of a Superpave mix design, along with specific aggregate selection and binder grade to enhance rutting resistance and ensure performance under local traffic and environmental conditions. The minimum Fine Aggregate

Angularity (FAA), as specified in AASHTO T304, varies with pavement thickness to ensure adequate structural stability and rutting resistance. For asphalt layers thinner than 100 mm, a minimum FAA of 45% is required because thinner pavements are more susceptible to surface deformation, and higher angularity helps create a strong interlocked aggregate skeleton within the limited thickness. For pavements thicker than 100 mm, the minimum FAA is reduced to 40%, as the additional structural depth provides greater resistance to applied loads, allowing slightly lower angularity without compromising overall mix stability. Similarly, the maximum value allowed for flat and elongated particles has been restricted to 10% and the maximum length-to-minimum thickness ratio allowed is 5:1. Sand equivalent is also restricted to 45% for fine aggregate. The fractured faces for coarse aggregates should be a minimum 95%. Compliance with all these properties would lead to a mix that interlocks well and is more resistant to permanent deformation.

The City of Vancouver also specifies performance-graded asphalt cements, which can be selected based on project locations and traffic loading requirements. The pavements are divided into two categories, including light-duty pavements for lower traffic loads and heavy-duty pavements for higher traffic loads and locations such as intersections and bus stops.

2.2.3 City of Calgary

The City of Calgary's approach to developing High Rut-Resistant mixes is also centred on the use of Superpave Asphalt concrete design methodology, tailored for high traffic and extreme climatic conditions. The specification emphasizes the use of Performance-Graded Asphalt Cement (PGAC) to match temperature conditions and traffic loads, ensuring optimal resistance to rutting. Table 2.1 shows the performance grade requirement for different road classes for the City of Calgary.

Table 2.1 Performance Grade Requirements (City of Calgary, 2021)

Application	Traffic (20-Year Design ESALs) million	Performance Grade
Base Course Lifts (All Roadway Applications)	N/A	PG 58-31
Surface or Binder Coarse (Residential and Non-Residential Transit Collector)	N/A	PG 58-31
Surface or Binder Course (Transit Collector)	N/A	PG 64-34
Surface or Binder Course (Major with no Intersections)	< 5	PG 58-31
Surface or Binder Course (Major with Intersections)	<5	PG 64-34
Surface or Binder Course (Major with no Intersections)	5 to 10	PG 64-34
Surface or Binder Course (Major with Intersections)	5 to 10	PG 70-31
Surface or Binder Course (Major with no Intersections)	>10	PG 70-31
Bridge Mastic Asphalt	N/A	PG 70-40

A higher performance grade is specified for surface or binder coarse with higher design ESALs; similarly, roads with intersections have been specified a higher performance grade for superior rutting performance.

2.2.4 City of Edmonton

The City of Edmonton specifies high rut-resistant asphalt mixes through a comprehensive framework that includes material selection, mix design, and performance testing. Performance Graded Asphalt Cement (PGAC), such as PG 58-28 and PG 64-28, has been specified with stiffer grades like PG 70-28 or PG 76-28 for more rut-resistant demanding applications. The City of Edmonton uses three distinct mixes, including 10mm-HT or High Traffic Mix for paving residential collector and selected arterial roadways. 10mm-LT or Low traffic Mix for paving low-

traffic residential roads and alleyways, and 20mm-B or Base course mix for heavy-duty applications such as freeways or industrial/commercial collectors. The specifications also mandate the use of the Bailey method to improve the aggregate skeleton for all three mixes and specify the Chosen Unit Weight (C.U.W) for coarse aggregate. The specification also specifies the consensus properties of aggregate to promote better aggregate selection, including fine aggregate angularity, abrasion loss, crushed face count, and soundness. Rutting susceptibility testing is also specified in the specification; all the mixes produced as per the specification should be subjected to the Asphalt Pavement Analyzer (APA) test, which ensures that the produced specimen meets the criteria for rutting resistance and results in a more durable mix. The specific requirement for the Bailey method and APA cut off criteria for rut depth are shown in Table 2.2 below:

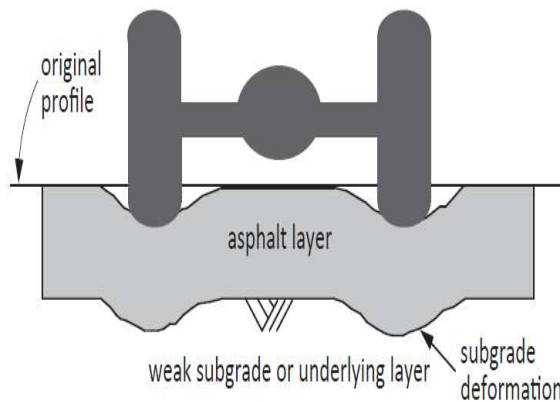
Table 2.2 CUW and APA requirement for Bailey method (City of Edmonton, 2023)

Parameters	10mm - HT	10 mm - LT	20mm - B
Bailey CA-CUW	60 to 85 Max - Fine Graded >95 to 105 Max - Course Graded	60 to 85 Max.	60 to 85 Max.
APA (mm, 52°C, 8,000 cycles)	5.0 max.	7.0 max.	5.0 max.

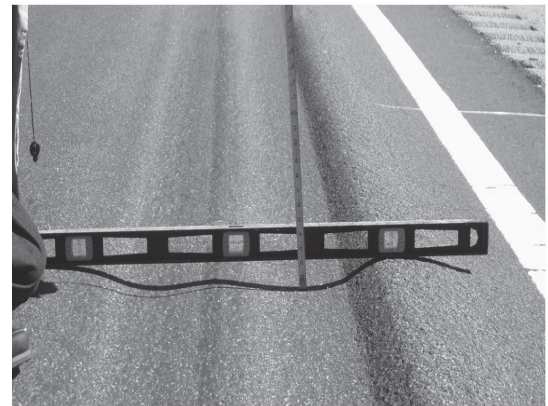
In summary, the comparative review of municipal asphalt specifications demonstrates that several jurisdictions have adopted progressive measures aimed at improving rutting performance and long-term pavement durability. While the City of Winnipeg's current specification provides a solid foundation, the gradual integration of these additional requirements could lead to enhanced quality control, more resilient mix designs, and improved service life of municipal pavements. This comparison not only highlights areas for potential refinement within Winnipeg's framework but also emphasizes the importance of continual benchmarking against peer municipalities to ensure the specification evolves in line with best practices.

2.3 Types of Rutting

Rutting in asphalt pavement is a long-standing issue that has persisted since the inception of asphalt pavements. It is one of the most severe forms of pavement distress, significantly affecting the long-term performance of the pavement and often leading to premature failure. According to the Asphalt Institute (2014), there are two primary types of rutting in asphalt pavements. The first type occurs when deformation primarily happens in the underlying layers, such as the base or sub-base (as shown in Figure 2.2(a) and is attributed to poor subgrade quality or inadequate layer thickness. The second type, known as instability rutting, results from an unstable asphalt mix, with the deformation confined to the asphalt layer (as shown in Figure 2.2b). This form of rutting is more common, and this research focuses on mitigating this specific type of rutting behaviour.



(a) Rutting from weak Subgrade (Asphalt Institute, 2014)



(b) Rutting from weak mixture (Asphalt Institute, 2014)

Figure 2.2 Types of Rutting

2.4 Mechanism of Rutting Accumulation

To mitigate rutting, it is important to understand the mechanism through which rutting accumulates (Du et al., 2018). The overall rutting depth in the pavement consists of three primary modes: material loss, densification, and lateral plastic flow. Material loss is often associated with a less durable mixture and is often characterized by ravelling along wheel paths. This mode has a minor impact on total rut depth; however, densification and plastic flow are the main factors in rutting accumulation. Densification occurs early in pavement life, particularly in mixes with higher air void content that are more prone to rutting. Over time, shear-induced deformation becomes a dominant factor contributing to long-term rutting. The relationship between permanent strain, permanent strain rate, and number of loading cycles applied to asphalt is shown in Figure 2.3. From the graph, it can be observed that during the initial loading cycle, the strain rate is high, which can be linked to the initial compaction and densification of the material as the voids between aggregates are gradually reduced. During this phase, asphalt adjusts to the load and has a low value of permanent strain as it recovers from deformation. In the stationery or middle phase, the strain and strain rate remain relatively constant, which suggests that the material has stabilized and can withstand additional load cycles without an increase in deformation. In the final phase, the strain rate increases, which leads to rapid deformation. This is due to the breakdown of the internal structure of the material; the asphalt mix becomes unstable, and as a result, deformation increases; hence, this is the phase where rutting or permanent deformation leads to failure of the pavement due to accumulated damage.

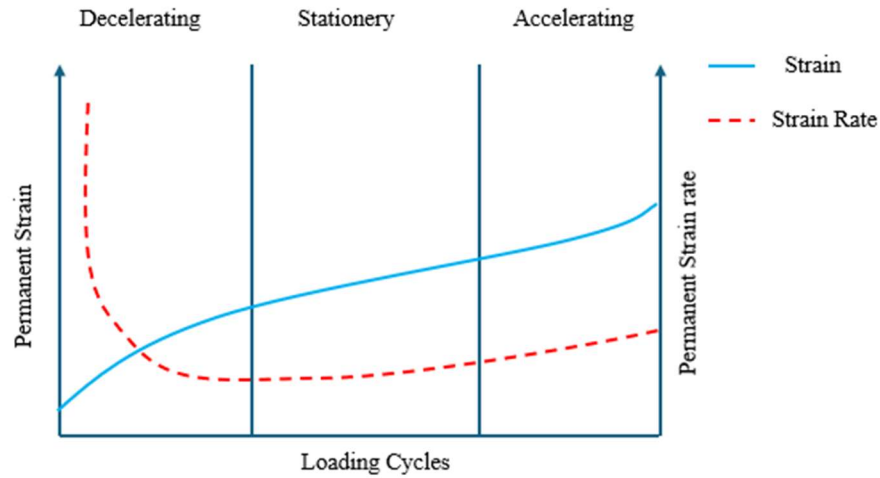


Figure 2.3 Strain and loading cycles relationships (Du et al., 2018)

2.5 Asphalt mixture behaviour across temperature ranges

Huber (1999) reported the response of the asphalt mixture across different temperatures. Asphalt mixtures exhibit nonlinear elastic, linear elastic, and viscous behaviour, and their performance varies with temperature and loading rate, as shown in Figure 2.4.

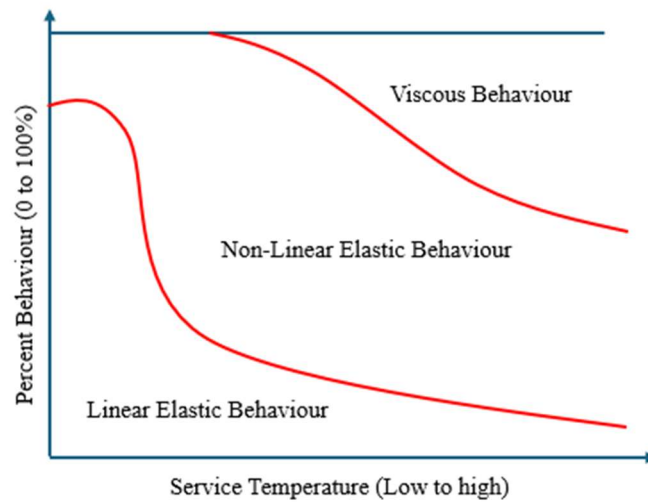


Figure 2.4 Variation in asphalt mixture behaviour with temperature changes (Huber, 1999)

At low temperatures in winter, asphalt is primarily elastic, and most energy from traffic loads is recovered through elastic rebound. This prevents rutting, and damage due to fatigue is minimal. In mid-range temperatures (spring or fall), the mix maintains some elastic properties but also displays viscous behaviour, although due to mid temperatures, stiffness is still relatively high to inhibit rutting; however, fatigue damage may occur under traffic loading. During the summer, the asphalt mix becomes more viscous, with reduced stiffness, and therefore, more rutting is observed. However, if strains remain relatively low, rutting is typically not severe. Excessive strains or a mix dominated by viscous characteristics can lead to significant rutting.

2.6 Characteristics of Rut-Resistant Mixes

Asphalt concrete is a composite material consisting of two primary components, i.e., asphalt binder and aggregates. Material properties, together with mix design and construction practices, influence how the mechanical response of asphalt mixtures varies with temperature (Huber, 1999). Several

factors impact the asphalt mix's rutting resistance internally, whereas some external factors also affect the rutting performance of HMA (Guo et al., 2020).

This section reviews different properties that can affect the performance of an asphalt mix with respect to rutting.

2.6.1 Aggregates

Aggregates are a fundamental component of Hot Mix Asphalt (HMA), which significantly influences its performance in terms of rutting resistance. HMA specifically constitutes up to 93% of aggregate by weight and around 85% by volume (Roberts et al., 1996). The characteristic of aggregates, including gradations, angularity (particle shape and texture), and absorption, plays a key role in determining a mix's mechanical properties and stability. An optimized gradation can result in a dense and interlocked structure, which is important for effective load distribution and to minimize deformation. The shape and angularity of aggregates are also significant factors that can promote interparticle friction and increase the mixture's strength. Coarse aggregates also have a pivotal role in the rutting characteristics of a mix as they provide structural stability and improve the high-temperature performance of the mix. Variations in aggregate properties can impact the pavement's compaction, stiffness, and overall durability. This underscores a need for immaculate aggregate selection and quality control of aggregates in the design of rut-resistant asphalt mixtures (Fang et al., 2018).

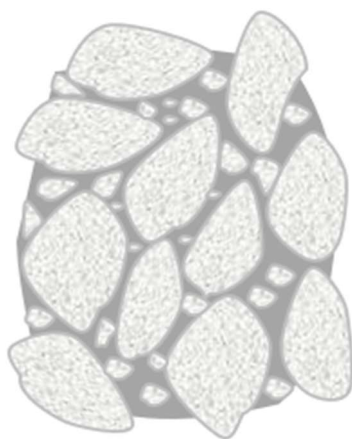
2.6.1.1 Aggregate Gradation

Aggregate gradation also plays a key role in the rutting resistance of the asphalt mix. A lot of research has been conducted on this area, highlighting the importance of gradation selection in the performance of HMA. Chen and Liao (2002) investigated how aggregate gradation influences

rutting performance and found that controlling the proportion of fines passing the 4.75-mm sieve is important. In their study, stone-on-stone contact developed at approximately 40% passing for a 19mm mix, while for a 9.5 mm mix, it began near 30% passing. The mixes with fine content within this range exhibited better rut resistance. Along with controlled portion of fine aggregates, the proportion of coarse aggregate in the mix plays a key role as it provides the aggregate skeleton and contributes significantly to internal friction, which is an important factor in resisting shear forces. Similarly, Meng et al. (2005) also highlighted the significance of limiting the fine particles in the mix to improve the rutting performance of HMA. In their study, they observed that high-temperature stability was reduced when the content of fine aggregates smaller than 2.36mm was increased in the mix. Yang (2007) also observed that a higher proportion of fine aggregates negatively affects the rutting resistance of asphalt mixtures. However, the study highlighted that simply increasing the coarse aggregate content does not always lead to improved performance.

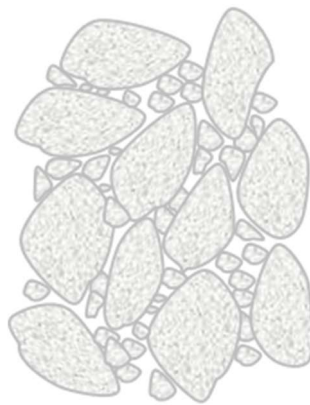
To understand the effect of gradation on asphalt mix performance, it is important to review different types of gradations used to develop HMA. Fang et al. (2018) and Memphis (2016) have summarized different gradation types in their studies. According to their studies, aggregate gradation can be classified based on morphology as either continuous or gap-graded, or based on air void content as dense, open, or semi-open graded. Continuous gradations incorporate a well-proportioned range of particle sizes, where smaller particles fill the voids between the larger aggregates, creating a compact and stable structure. This dense gradation is typically used in Asphalt Concrete (AC) and Asphalt Treated Base (ATB). It minimizes moisture infiltration while maintaining sufficient air voids (3-6%) to accommodate thermal expansion for the asphalt binder. On the contrary, gap gradation lacks certain intermediate particle sizes, resulting in a coarser aggregate skeleton dependent on stone-to-stone contact for the stability of the mix. Stone Mastic

Asphalt (SMA) exemplifies this type of gradation, which provides excellent rutting resistance due to a strong interlocking framework. Semi-open, open gradations, and Open-graded Friction Courses (OGFC) prioritize permeability but may sacrifice some rutting resistance. Uniform gradation is another gradation type in which particle sizes are constrained in a narrow range. This type of gradation is normally used in Portland cement concrete and is not generally applied to asphalt mixtures. Figure 2.5 shows the schematic diagram of different types of gradations.



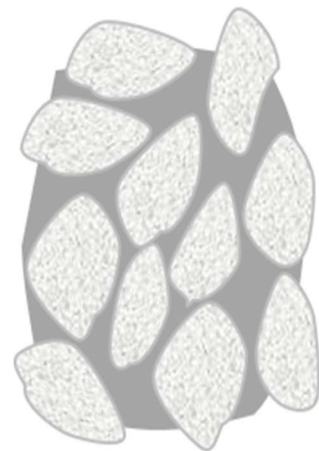
(a)

Wide range of sizes
Stone to stone contact
Low Permeability



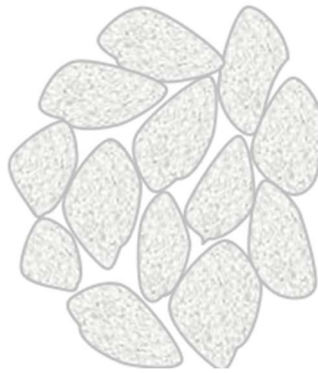
(b)

Wide range of sizes
Stone to stone contact
High Permeability



(c)

Intermediate sizes missing
Moderate Permeability
No Grain-to-grain Contact



(d)

Narrower size range

High Permeability

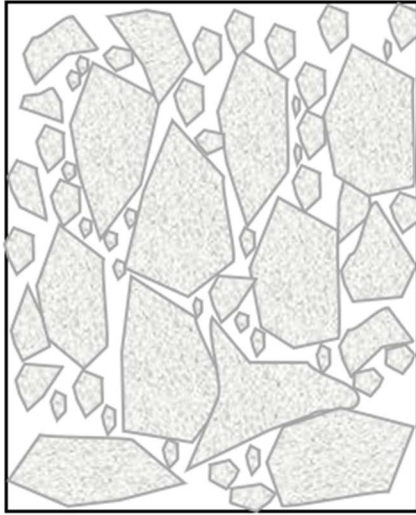
Difficult Compaction

Figure 2.5 Four Common Types of Aggregate Gradation (Memphis 2016): (a) Dense Gradation, (b) Open Gradation, (c) Gap Gradation, (d) Uniform Gradation.

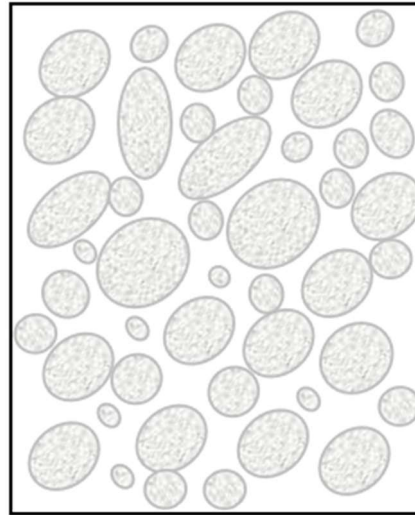
Dense-graded aggregates include a variety of particle sizes, ensuring grain-to-grain contact and lower air void content in the mix. The benefits of this gradation include reduced permeability, improved stability, and increased resistance to rutting under traffic loads. It is important to note that while this aggregate structure is ideal for high rut-resistant mixes, achieving effective compaction can be challenging because the close packing of coarse and fine particles limits aggregate mobility during compaction. As a result, higher compaction effort is often required to adequately reorient particles and achieve the target density without over-crushing or segregating the mix. Hajj et al. (2011) highlighted the significance of aggregate structure integrity in improving the rutting resistance of Hot Mix Asphalt (HMA). The Bailey Method, introduced by Robert D. Bailey in 1997, provides a systematic approach to enhance the aggregate structure. This method enhances shear strength and improves rutting resistance in HMA pavements by promoting interlock among coarse aggregates and forms a dense, well-structured skeleton. This method can be applied to improve the mix's stiffness and, consequently, its overall rutting performance.

2.6.1.2 Aggregate Angularity

In asphalt mixtures, aggregates typically make up 94 to 96% by mass, with fine aggregates comprising about 40% of the total aggregate mass. The quality and quantity of both fine and coarse aggregates play a significant role in the performance of HMA, including stiffness, stability, durability, permeability, moisture susceptibility, rutting resistance, and air voids. The geometric irregularity of aggregates, such as particle shape, angularity, and surface texture, also influences these properties. Round or smooth-textured and nonporous aggregates tend to increase susceptibility to rutting, while angular or rough-textured and slightly porous aggregates improve resistance to rutting. Angular particles, commonly found in crushed stone, provide better interlocking and shear strength than rounded aggregates. (Ramli et al., 2013). Asphalt Institute (2014) also emphasized that resistance to permanent deformation in asphalt mixtures is primarily influenced by the internal friction of aggregate particles and, to a lesser extent, by the cohesion provided by asphalt binder. Internal friction depends on the shape, surface texture, and gradation of aggregates. Shear strength in asphalt mixture can be further enhanced by selecting angular, rough-textured particles with proper gradation to enhance inter-particle friction and improve overall stability. The difference between angular and rounded aggregates is shown in Figure 2.6.



(a) Angular Aggregates



(b) Rounded Aggregates

Figure 2.6 Aggregate Framework Asphalt Institute (2014)

Some other studies also highlighted the importance of aggregate angularity for the rutting resistance of asphalt mix. Button et al. (1990) identified excessive fine aggregates and rounded particles as key factors in rutting. Studies have shown that with their angularity and rough textures, crushed gravel aggregates enhance the stability and strength of asphalt mix, reducing rutting and improving water resistance. (Herrin, 1954; Lottman & Goetz, 1956; Wedding & Gaynor, 1961)

Kalcheff and Tunicliff (1982) further confirmed that a mixture with crushed coarse and fine aggregates exhibits higher resistance to permanent deformation compared to those with natural sand. These findings further underscore the importance of aggregate geometric angularity in ensuring pavement performance. Yasreen et al. (2011) found that higher Fine Aggregate Angularity (FAA) values, more angular and rough-textured particles, result in better interlocking and shear strength. Shu et al. (2006) studied the influence of Coarse Aggregate Angularity (CAA) on the rutting performance of Hot Mix Asphalt (HMA). Their findings demonstrated that CAA

significantly affects rutting performance, with more angular coarse aggregates leading to reduced rut depths and enhanced rut resistance.

2.6.1.3 Aggregate Size

Aggregate size plays a crucial role in enhancing the rutting resistance of asphalt mixtures. Brown and Basset (1990) examined the performance of mixes with varying maximum aggregate sizes and found that mixtures containing larger aggregates exhibited greater strength compared to those prepared with smaller aggregate sizes. A similar study carried out by Kim et al. (2018) on HMA mix with RAP revealed that the rut depth increased as the Nominal Maximum Aggregate Size (NMAS) decreased. A study conducted by Hafeez et al. (2012) on the effect of the maximum size of aggregate on the rutting potential of Hot Mix Asphalt also revealed that rutting decreases with an increase in nominal aggregate size. In the study, mixes with four NMAS were evaluated, ranging from 25.4mm to 9.5mm. 25.4mm NMAS mix exhibited the least rut depth at 10,000 and 60 °C; the overall rut depth was 6.6mm, in contrast with the 9.5mm NMAS mix that showed 15.14mm rut depth at the same number of cycles and temperature.

2.6.2 Asphalt Binder

The asphalt binder has a significant influence on the rutting performance of the asphalt mix. The two main factors that affect the mix performance are the stiffness of the binder and the asphalt content. Optimal asphalt content in an HMA ensures adequate binding and flexibility, which is critical for load distribution and resistance to permanent deformation. However, deviations from optimal content, either excess or insufficiency, can adversely affect the pavement's performance. Brown and Cross (1989), in their study, emphasized that rutting in asphalt pavement is often caused by excessive asphalt content. To prevent this, the asphalt content should be carefully

selected and controlled during production. Increasing asphalt content arbitrarily should be avoided for improving compaction, reducing segregation, or other workmanship issues; it should only be adjusted to achieve the desired air void content for durability and performance. In contrast, asphalt mixtures with insufficient binder content do not provide enough material to properly coat the aggregate particles, leading to reduced workability and durability. Durability of HMA refers to the ability of the mix to resist cracking caused by brittleness and susceptibility to moisture-induced damage, such as stripping. (Pan et al., 2023). Monismith and Tayebali (1988) revealed that polymer-modified binders offer significantly better rutting resistance at high temperatures than unmodified ones. They also found that adding polymers increases the binder's viscosity at elevated temperatures, improving its rutting resistance without compromising performance at lower temperatures.

2.6.3 Mixture Volumetric Properties

2.6.3.1 Air Voids

Air voids in an asphalt mix represent the small pockets of air trapped within the compacted pavement. Optimal air void content is essential for ensuring the balance between durability and flexibility. Suo et al. (2009) emphasized that air void plays an important role in the rutting behaviour of asphalt mixtures under wheel loads. To limit permanent deformation, the airvoid content of the asphalt mixture must be kept closer to 4.5%; anything below that range tends to increase the susceptibility of mixes to rutting. Mixes subjected to lower compaction effort, having higher air void content than 7-8% can increase rutting susceptibility as well. Brown and Cross (1989) also reported that HMA mixes with voids lower than 3% are highly prone to rutting. Zaltuom (2018) also emphasized that at very low air voids (below 2%), the binder fills the space between aggregate particles; as a result, the mixture behaves more like a fluid and becomes highly

susceptible to deformation under heavy traffic. Roy et al. (2013) reported that mixes with higher air voids undergo rutting due to densification during the initial period of traffic as pavement deforms under traffic loads. A study conducted by Zhang et al. (2018) showed that both the porosity level and spatial distribution of air voids in asphalt concrete significantly affect its resistance to rutting. To enhance the rutting resistance of asphalt concrete, it is important to ensure that porosity meets the target design level and that air voids are evenly distributed throughout the mixture. Verification of the target porosity level can be carried out through the determination of air voids through conventional volumetric testing; however, the spatial determination of air voids requires imaging-based assessment. Techniques such as X-ray scanning, digital image analysis of sawn specimen faces and field GPR measurement allow assessment of void distribution within the mix. Guo et al. (2020) also highlighted that insufficient air voids weaken the aggregate interaction, reducing bearing capacity and rutting resistance. Conversely, excessive air voids create a loose structure, prevent proper aggregate interlocking, and limit asphalt binder coverage, which compromises the mixture's stability.

2.6.3.2 VFA and VMA

VMA is the void volume between the aggregate particles in a compacted asphalt mixture, including air voids and the space filled by the binder. A higher VMA allows for adequate binder content, enhancing the mix's durability and resistance to permanent deformation. However, excessive VMA can lead to an overly binder-rich mix in case of higher binder content, making pavement prone to rutting under heavy loads (Roberts et al., 1996). Field investigations conducted by Li et al. (2013) revealed that the actual Voids in Mineral Aggregates (VMA) and air voids are often lower than the recommended minimum levels due to high stress and braking forces. This deficiency makes the Hot Mix Asphalt (HMA) more prone to rutting as the pavement becomes

plastic and unstable under traffic loads. Maintaining adequate VMA is critical for ensuring sufficient space to accommodate asphalt binders, which improves rutting resistance, particularly in high-stress areas like intersections. Excessive VMA can negatively impact the ability of HMA to resist deformation due to rutting. Huber and Heiman (1987) recommended that Voids Filled with Asphalt (VFA) be limited to a maximum of 70% to minimize rutting. VFA is directly related to the balance between binder and aggregate structure; a higher value of VFA indicates that more space within the aggregates is filled by asphalt binder, which results in a reduction of the mix's internal friction and weakens the aggregate interlock.

2.7 Traffic Characteristics

Traffic characteristics also have a significant impact on the rutting performance of asphalt mixes. Tire contact pressure is one of the major factors that may influence the extent of rutting on a pavement, along with vehicle speed and frequency of load application. The shape of the tire-pavement contact area is nearly circular under relatively small wheel loads; however, as the wheel load increases, while the inflation pressure remains constant, and the contact area becomes progressively elongated (Albayati, 2023). This phenomenon is illustrated in Figure 2.7.

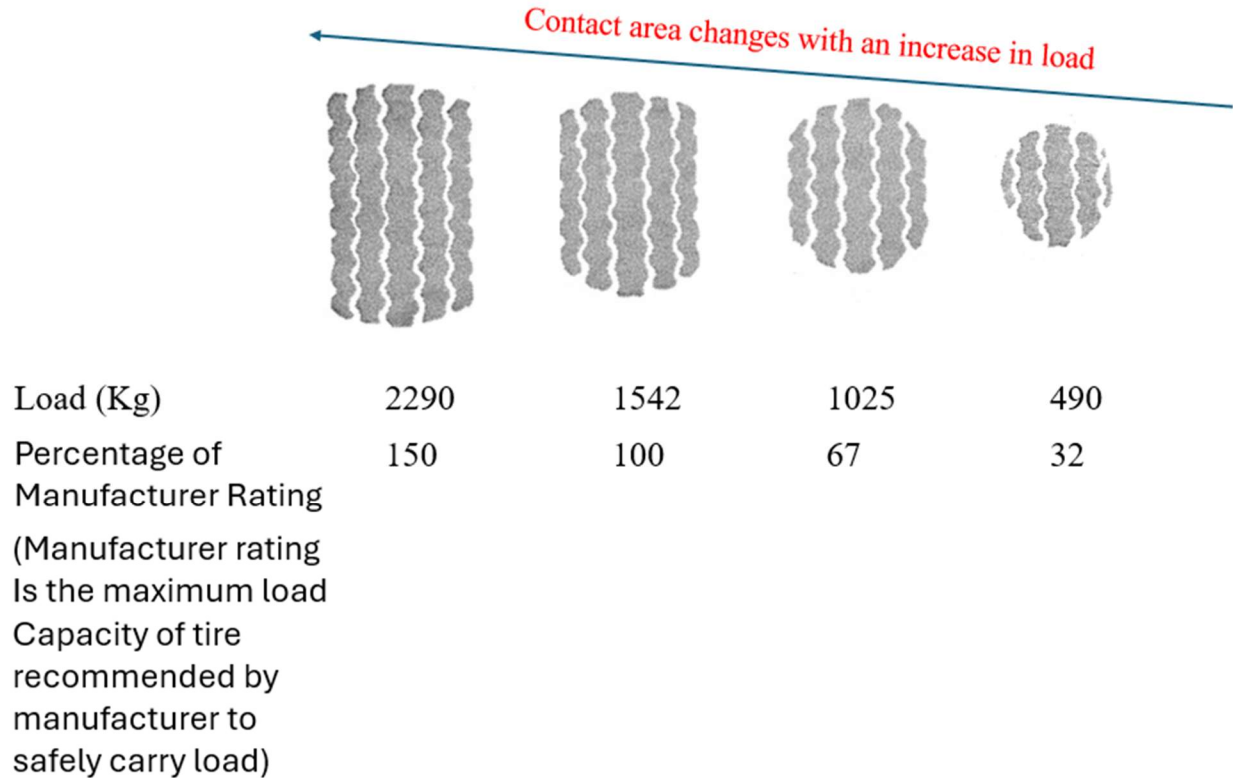


Figure 2.7 Tire Prints of Different Loadings (Albayati, 2023)

In the above figure, it can be observed that in the heaviest loading condition, the manufacturer's tire rating is exceeded by 50%. At higher loads than rated capacity, the contact area shape changes (more elongated than circular), which causes non-uniform stress distribution, leading to higher pavement stresses and more rutting.

Since the original AASHTO road test, there has been a rise in both truck axle weights and tire pressures, which has contributed significantly to increased rutting in hot mix asphalt (HMA) pavement (Betenson et al., 1984). The 1993 AASHTO design guide still bases pavement design on 18-Kip loads with tire contact pressure between 75-80 psi. However, as per some of the recent studies, the truck tire pressures are now inflated to about 100 psi, with some reported cases

reaching as high as 140 psi (Middleton et al., 1986; Al-Mosawe et al., 2016). Field studies by Chen et al. (2000) using the Texas load simulator revealed that increasing tire pressure from 80 to 100 psi could increase rut depths by 43% after 1.6 million axle load applications.

Vehicle speed is another traffic parameter that strongly influences the permanent deformation of asphalt pavements. Generally, higher speeds correspond to higher effective moduli and lower pavement deformations (Croney et al., 1991). Pereira et al. (1999) investigated rut development on steep highway grades in Portugal. At the start of the grade, the rut depth averaged 10mm, but at the upper end (5% slope and 25 km/h average speed), where the speed reduced, the rut depth increased to 20mm. These studies indicate that the cumulative number of load applications is directly proportional to the severity of rutting. As the frequency of axle load repetition increases, rut depth also increases, making traffic volumes one of the most important factors affecting the rutting of pavements.

2.8 Moisture-Induced Damage in Asphalt Pavements.

In addition to rutting, moisture-induced damage is a major factor influencing the long-term performance of asphalt mixtures. These mixtures are generally composed of binders, aggregates, and occasionally modifiers. While binder content has been previously highlighted for its role in rutting resistance, its primary function is to form a protective film around the aggregates, thereby safeguarding them against moisture infiltration (Neupane & Wu, 2025). Moisture resistance is an important property for the durability of the asphalt mixture. When moisture penetrates the mixture, it weakens the adhesive bond between binder and aggregates, initiating a stripping process that deteriorates pavement structure. This reduction in structural integrity often leads to distress, such as rutting, cracking, and pothole formation, which necessitate frequent maintenance interventions.

Moisture damage in asphalt mixtures is typically categorized into two primary failure modes. The first is adhesive failure, which occurs when the bond between the binder and the aggregate surface breaks, leaving the aggregate clean and exposed. The second is cohesive failure, which takes place within the binder–mastic itself, where the internal structure of the binder and filler breaks down, resulting in visible distress (Soenen et al., 2020; Nejad et al., 2012). The severity of moisture-induced damage is influenced by several factors, including aggregate and binder types, mixture designs, construction methods, environmental conditions, traffic loading, and use of antistripping additives (Stuart, 1990). Moisture damage is also greatly influenced by aggregate type. Apeagyei et al. (2015) investigated the susceptibility of asphalt binder and aggregates systems to moisture-induced damage using the Binder Bond Strength (BBS) test. The study compared different types of aggregates, including granite, limestone, and Basalt, combined with a 40/60 penetration grade binder. Results of the study revealed that granite exhibited greater moisture-related damage than limestone, which was attributed to its higher absorption and diffusion characteristics. Similarly, Copeland et al. (2007) conducted a study on the moisture damage potential of asphalt binders PG 52-34, PG 64-28, and PG 70-22, both unmodified and eight different modified binders. The study showed that the use of polymer-modified binders does not consistently improve resistance to moisture damage. Figure 2.8 shows the mechanism of adhesive and cohesive moisture-induced damage.

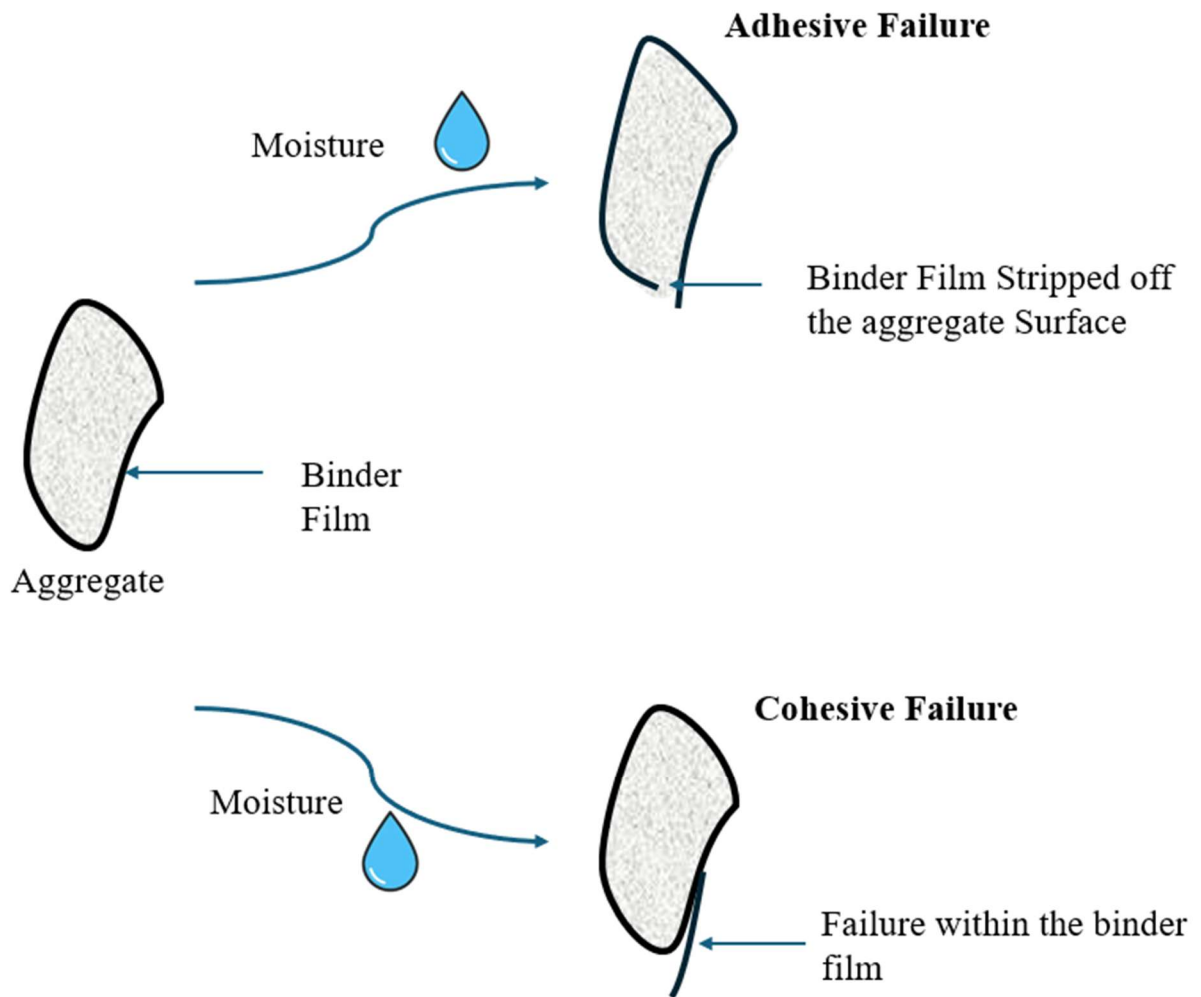


Figure 2.8 Adhesive and Cohesive Failure Due to Water Damage (Ameri et al., 2021)

Chapter 3. Methodology, Materials, and Testing Program

3.1 Methodology

City of Winnipeg has traditionally utilized the Marshall mix design method for most of its in-service pavements, primarily incorporating locally sourced limestone and gravel aggregates. This study, as outlined earlier, focuses on the development and evaluation of Superpave mixes designed with high-quality granite and basalt aggregates. While previous research has extensively highlighted the superior performance of Superpave mixes over Marshall mixes, this study provides a localized perspective, offering insights into the performance of Superpave mixtures when developed using aggregates available within the Winnipeg region.

Laboratory-produced mixes provide greater control over material selection, aggregate proportioning, and mix design parameters, enabling the development of a consistent and performance-optimized asphalt mixture. In contrast, plant-produced mixes are more susceptible to variations in volumetric properties due to factors such as high production volumes, equipment calibration limitations, variability in stockpile uniformity, and temperature fluctuations during transportation and placement. Considering these differences, the experimental program for this study was designed to evaluate both laboratory and plant-produced mixes. Specifically, one laboratory-produced mix was developed, compacted, and tested entirely under controlled laboratory conditions, while five plant-produced mixes, supplied directly from the field, were compacted and tested in the laboratory without modification.

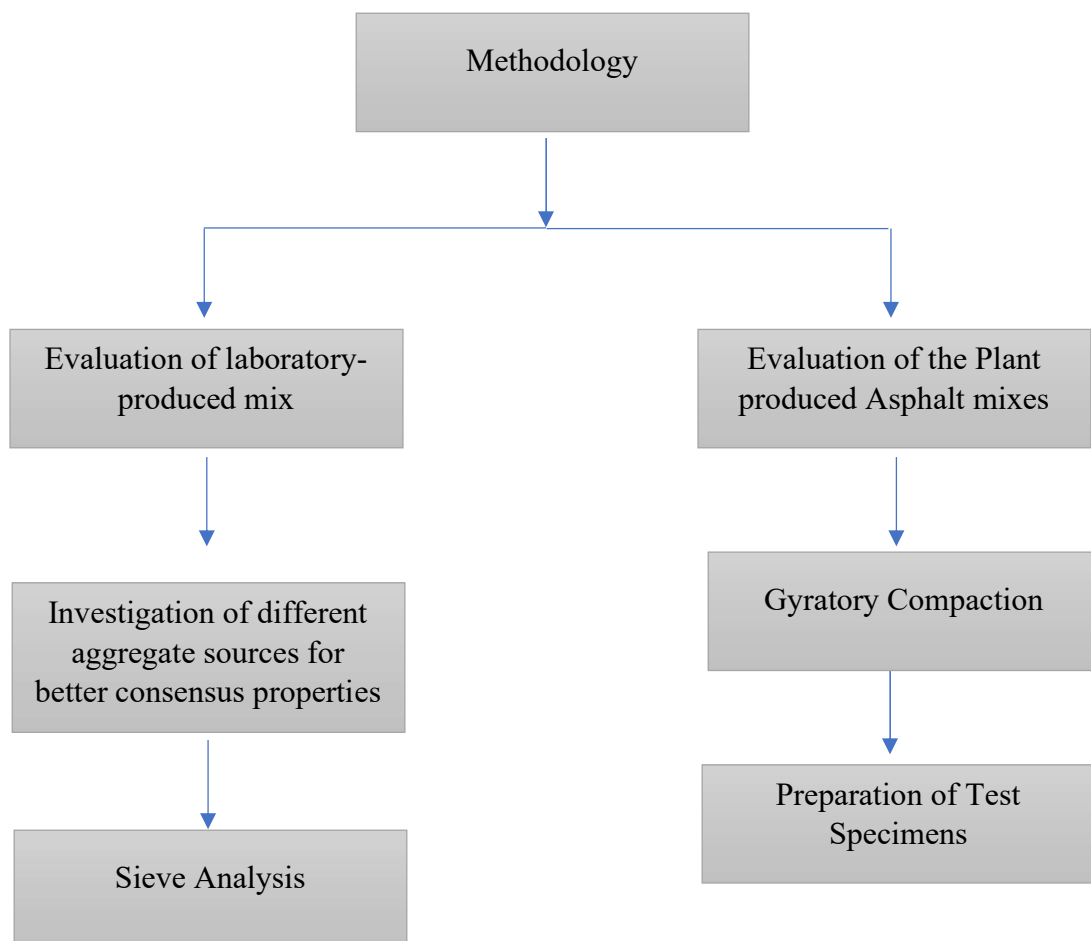
For the development of laboratory mix, a higher emphasis was placed on the investigation of aggregate sources with an aim of selecting the aggregates with superior mineralogy, angularity, and other relevant properties, as explained later in this chapter. The quality of aggregates alone is

not adequate to provide resistance against permanent deformation, as discussed earlier; gradation of the aggregate blend plays an important role in creating a mix stiff enough under high traffic loads. The goal was to achieve a gradation that is coarser in nature and has a limited amount of fine particles, although this would have entailed having a relatively higher binder content in the mix to compensate for the lack of fines; but overall stiffness of the mix was expected to increase. The aggregate skeleton of the laboratory-produced mix was further improved by incorporating Bailey's method, which involves checking the coarser and finer portions of aggregates and limiting them to specified proportions. The optimum binder content for the laboratory-produced mix was the highest among Superpave mixes evaluated under this study, which was determined by compacting three different trial mixes with increasing binder content for each trial. The laboratory-produced mix was evaluated for rutting resistance, moisture susceptibility, and stiffness characteristics using the Hamburg Wheel Tracking Test (HWTT), Moisture Sensitivity Test, and Dynamic Modulus Test, respectively.

Concurrently, five plant-produced loose mixes were obtained from five different pilot projects across the city. Among these, four mixes were designed using the Superpave mix design method, while one mix followed the Marshall mix design. The mixes were designated as Project 1 SP1, Project 1 SP2, Project 2 SP1, Project 2 SP2, and Project 3 MS1. The SP1 and MS1 mixes were collected from the surface course (12.5 mm NMAS), whereas the SP2 mixes were obtained from the binder course (19 mm NMAS). These mixes were compacted in the laboratory to prepare specimens for performance testing.

All five plant-produced mixtures were evaluated for rutting resistance, moisture susceptibility, and stiffness characteristics using the Hamburg Wheel Tracking Test (HWTT), Tensile Strength Ratio (TSR) test, and Dynamic Modulus Test (DMT), respectively. As noted earlier, large-scale

production and transportation processes can sometimes lead to variability in the volumetric and material properties of plant-produced mixes. To verify whether the binder content and aggregate angularity of the supplied mixes were consistent with the design specifications, a forensic investigation was carried out on the Project 1 and Project 2 mixes. Two specimens from each mix were tested using the ignition oven method (AASHTO T308) to determine the actual binder content by measuring the mass loss at elevated temperatures. The recovered aggregates were then subjected to the Fine Aggregate Angularity Test (AASHTO T304) to compare the measured angularity of the supplied mixes with the original design values. This evaluation provided valuable insights into the variation in performance of plant-produced mixes across different testing conditions. Figure 3.1 Outlines of the methodology adopted to achieve the objectives of this research.



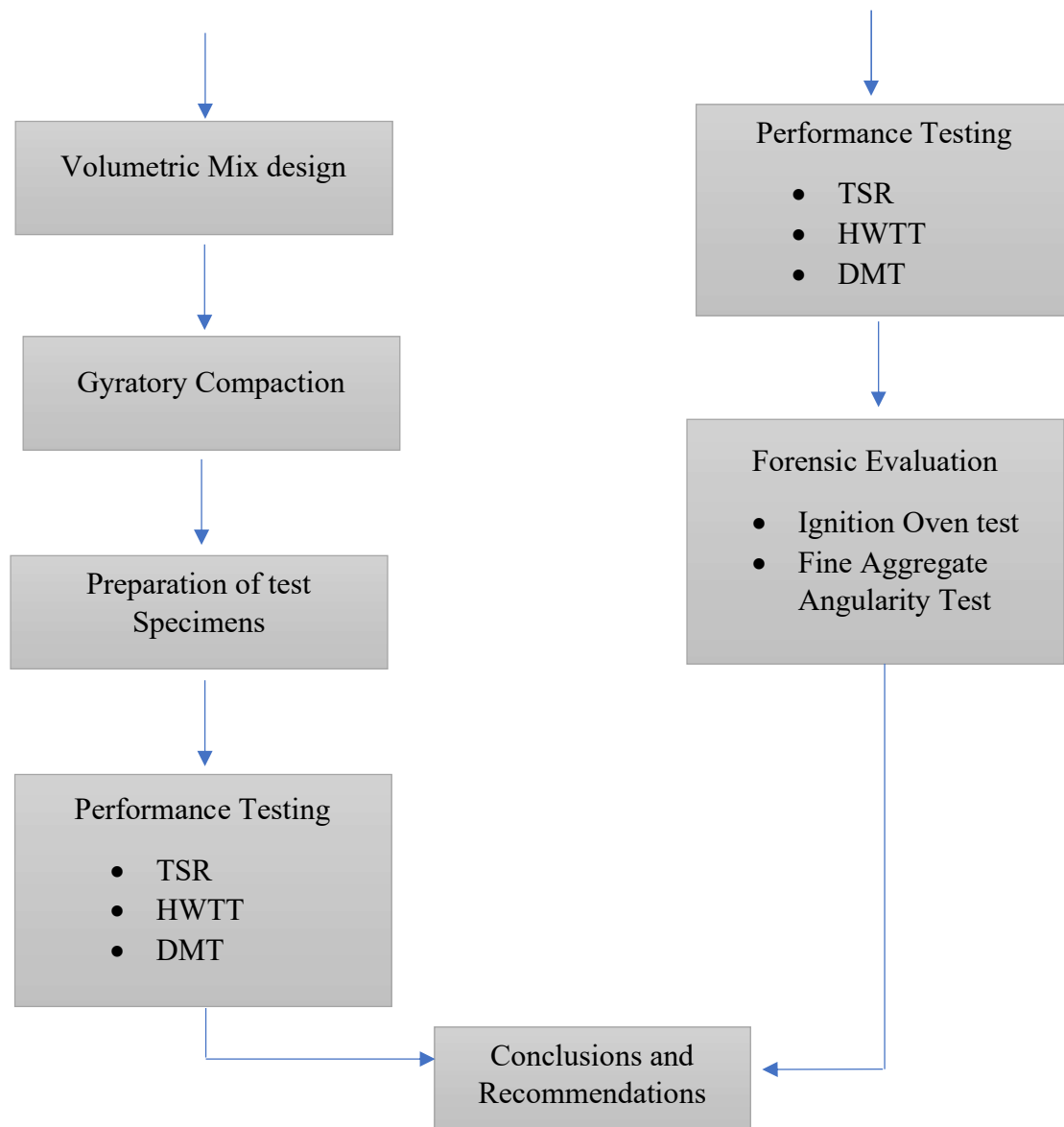


Figure 3.1 Methodology for evaluating rutting potential and moisture sensitivity of the mixes

3.2 Materials

3.2.1 Aggregates Selection and Gradation for Laboratory-Produced Mix

Aggregate selection was an important part of this study. As discussed earlier, City of Winnipeg has conventionally utilized limestone and gravel sources for its mixes; moreover, the lack of any selection criteria in the current specification entailed that the aggregates selected for current projects were not screened for important properties such as Fine Aggregate Angularity (FAA), which are critical for better rutting performance of the mixes.

Aligning with the aim of this study, the first step was to investigate alternative sources of aggregates for mix development. As discussed in the literature review, aggregate mineralogy has a considerable impact on the performance of the mixes. Basalt and granite aggregates, when compared to limestone aggregates, are generally harder and therefore are expected to provide better resistance against permanent deformation. Limestone and gravel sources used in the current projects are located closer to the city, as shown in Figure 3.2. The alternative sources used in this study were at relatively longer distances from the city; the granite sources were located 75 km and 103 km away. The basalt-based aggregate quarry was at the farthest distance from the city, with an overall distance of 234 km.

The aggregates from these sources were supplied in bags, and each source was segregated into three distinct categories: coarse aggregates, which were predominantly larger aggregate sizes in the range of 16-9.5mm, and manufactured fines, which had a relatively wider aggregate size distribution. However, manufactured fines still contained a considerable proportion of coarser particles, including 9.5mm and 4.75mm, which were sieved out in the lab to keep the passing percentages within the city-specified range for the overall blend. The natural fines were mainly

fine aggregates without any significant portion of coarser range aggregates. The gradation for each source is shown in Table 3.1.

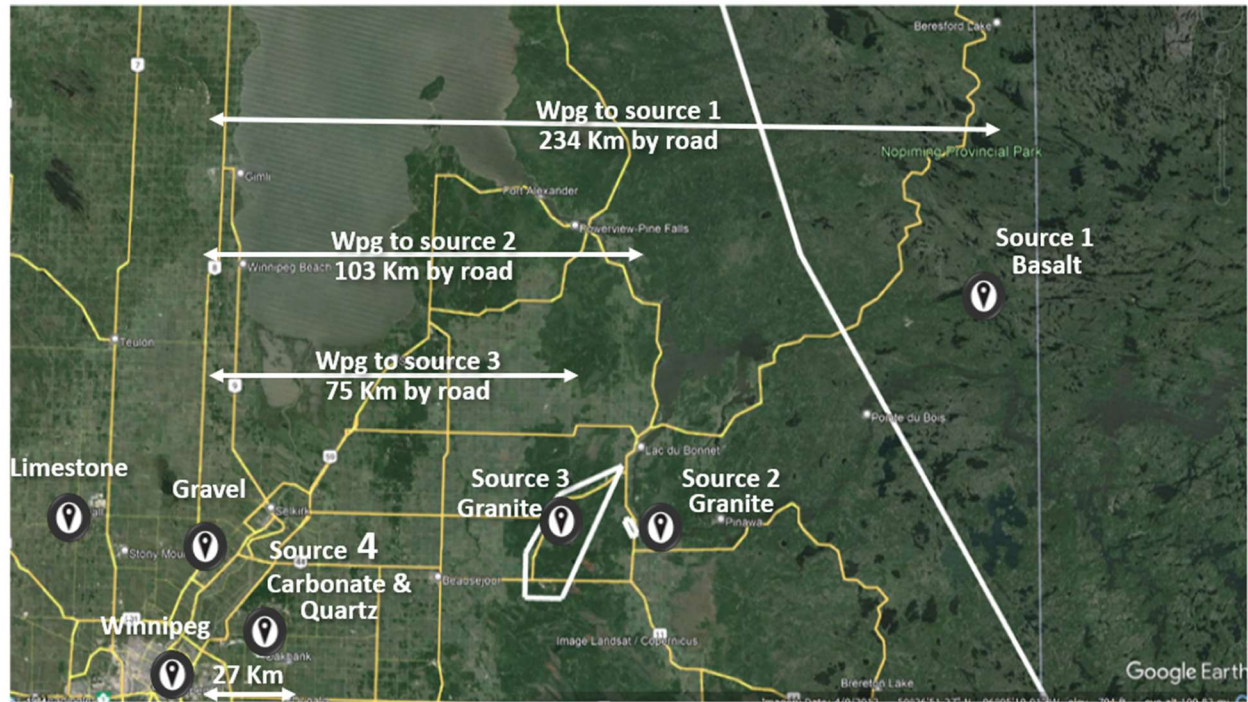


Figure 3.2 Location of Aggregate Quarries

Table 3.1 Gradation Results from Sieve Analysis (AASHTO T27, 2023)

Sieve Size (mm)	Source 1 Coarse (%)	Source 2 Coarse (%)	Source 3 Coarse (%)	Source 1 MF (%)	Source 2 MF (%)	Source 3 MF (%)	Source 3 NF (%)	Source 4 NF (%)
19	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00
16	92.99	94.16	98.48	99.87	100.00	100.00	100.00	100.00
12.5	52.33	38.45	45.53	99.80	100.00	100.00	100.00	100.00
9.5	9.74	8.21	9.17	99.48	99.05	99.22	100.00	100.00
4.75	0.32	0.58	0.59	65.55	60.46	86.57	65.55	93.84
2	0.32	0.58	0.59	37.66	34.37	64.71	38.52	79.26
0.425	0.32	0.58	0.57	14.57	14.16	12.84	16.26	24.09
0.18	0.28	0.55	0.47	9.06	7.48	3.43	8.94	2.63
0.075	0.23	0.52	0.31	5.84	3.18	0.78	4.17	0.45

As shown in Table 3.1, the NMAAS for coarse aggregate is 12.5mm, for the manufactured fines, it is 4.75mm, and for the natural fines, the NMAAS is 2.0mm. The difference in gradation of each source is also shown in Figure 3.3.

To develop mixes with high resistance to rutting, it was essential to evaluate the consensus properties of aggregates, as these properties indicate aggregate quality and its contribution to overall mix performance. Among the consensus properties, Fine Aggregate Angularity (FAA) is of particular importance because higher angularity in fine aggregates promotes better friction, reduces the tendency for particle slippage under traffic loading, and ultimately enhances the stiffness and rutting resistance of asphalt mixtures. In this study, the FAA test was conducted in accordance with AASHTO T304, which measures the uncompacted void content of fine aggregates as an indicator of angularity and surface roughness.

The results showed that Source 1 and Source 3 exhibited the highest uncompacted void contents at 50.64% and 49.19%, respectively as given in Table 3.3. These values are well above the minimum requirement of 40% specified in the City of Winnipeg (COW) specification CW 3410 R13, confirming that these sources possess a high degree of angularity and are thus expected to contribute positively to the rutting performance of the designed mixes. Conversely, other sources, while acceptable in terms of gradation, demonstrated relatively lower uncompacted void contents, making them less desirable from an angularity standpoint.

Similarly, coarse aggregates also have an important role in providing stability to the aggregate structure, as discussed in the literature review. Source 1 showed the lowest abrasion loss at 14.1% and the highest specific gravity at 2.75, as shown in Table 3.2. Lower LA abrasion reflects superior resistance to particle breakdown and shear deformation under traffic loads. Therefore, source 1 aggregate was selected for producing the Laboratory mix.

Based on these findings, Source 1 and Source 3 were selected as the most suitable fine aggregate sources for incorporation into the high rut-resistant mix designs. Their superior angularity was expected to improve internal aggregate friction and provide greater mix stability under heavy traffic conditions. This careful selection aligns with the broader objective of the study to move beyond conventional limestone and gravel aggregates by integrating harder and more angular mineralogy, such as granite and basalt, to achieve longer-lasting pavement structures with reduced susceptibility to permanent deformation.

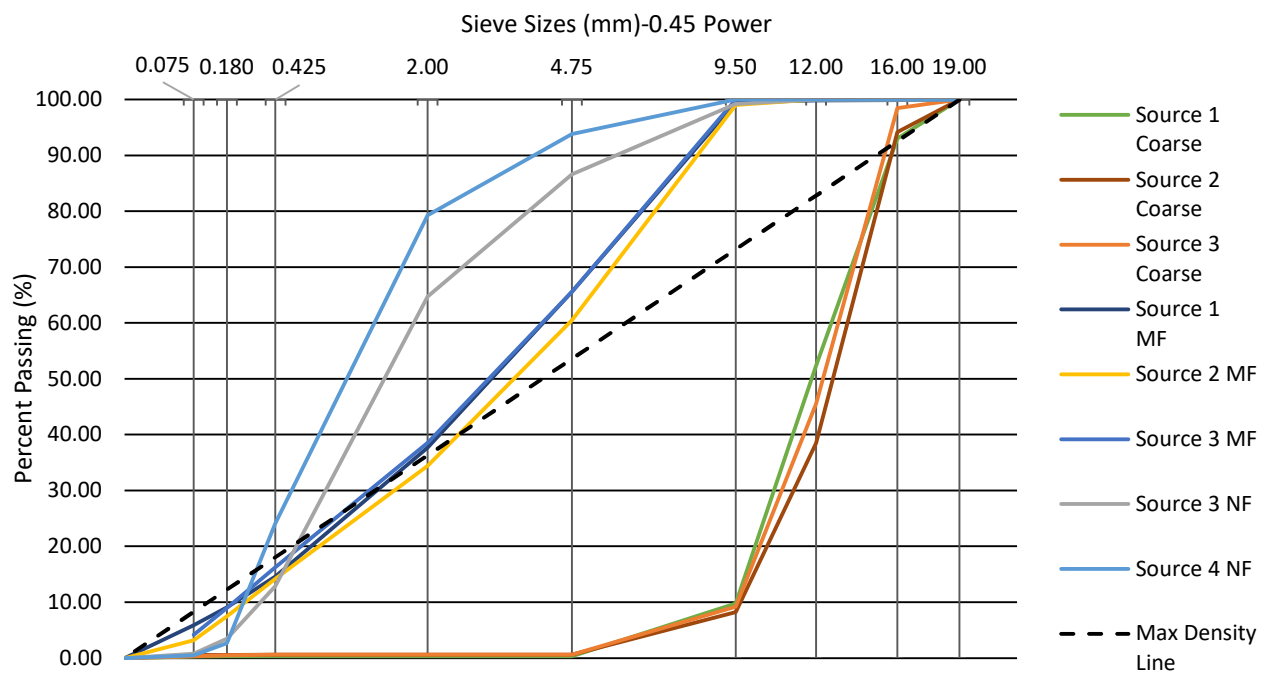


Figure 3.3 Comparison of aggregate sources

Table 3.2 Properties of Coarse Aggregates

Aggregate Source	LA Abrasion	Micro Deval	Absorption (%)	Specific Gravity	Uncompacted Voids (%)
Source 1 Coarse	14.1	-	0.50%	2.75	N/A
Source 2 Coarse	29.4	-	0.43%	2.62	N/A
Source 3 Coarse	23.0	8.4	0.58%	2.64	N/A

Table 3.3 Properties of Fine Aggregates

Aggregate Source	LA Abrasion	Micro Deval	Absorption (%)	Specific Gravity	Uncompacted Voids (%)
Source 1 (MF)	N/A	-	0.50%	2.75	50.64
Source 2 (MF)	N/A	-	0.43%	2.62	48.15
Source 3 (MF)	N/A	-	0.58%	2.64	49.19
Source 3 (NF)	N/A	-	0.70%	2.58	39.26
Source 4 (NF)	N/A	7.4	1.65%	2.62	40.76

The Bailey method was adopted to optimize the gradation of the laboratory-produced mix and to improve aggregate packing and limit the amount of fines for better rutting performance. Vavrik et al. (2002) outline the procedure to blend aggregates from different sources as per the Bailey method. As part of the procedure, the first step is to select a target unit weight for the coarse aggregate fraction of the mix. Based on the selected unit weight, the volume of voids in the coarse aggregate is determined, and these voids are filled by an appropriate quantity of fine aggregates using the rodded unit weight.

First, the loose unit weight of the selected coarse aggregate was determined using the shovelling procedure specified in AASHTO T 19. As per the standard, coarse aggregates are poured into a clean, dry container of known volume from a fixed height, allowing material to free fall. During the filling process, no compaction is applied, the container is overfilled, and then excess material is struck off using a straight-edge glass. The filled container is then weighed and unit weight is computed by dividing the mass of aggregates by the volume of the container. A summary of the

measured loose unit weight for coarse aggregates is presented in the table below. Since source 1 coarse aggregate was selected for the laboratory mix, the loose unit weight of only source 1 is given in Table 3.4 below.

Table 3.4 Loose Unit Weight of Aggregates (AASHTO T-19, 2014)

Coarse Aggregate Source	Loose Unit Weight (Kg/m ³)
Source 1	1289.50

Additionally, the rodded unit weight of the fine aggregate was determined using the rodding procedure outlined in AASHTO T-19. The compacted unit weight of the fine aggregate was then used to calculate the fine aggregate needed to fill the voids in the coarse aggregates. Rodded unit weight of fine aggregates laboratory-produced mixes are given in Table 3.5.

Table 3.5 Rodded Unit Weight of Aggregates (AASHTO T-19,2014)

Fine Aggregate Source	Rodded Unit Weight (Kg/m ³)
Source 1 MF	1626.88
Source 4 NF	1766.37

The final proportion of coarse aggregates was 43%, manufactured fines and natural fines were only 42% and 15% respectively.



(a) Rodded Unit Weight Sample



(b) Loose Unit Weight Sample

Figure 3.4 Unit Weight Procedure for Bailey Method

Table 3.6 shows the gradation achieved after applying the Bailey check; the percentage of coarse aggregates in the mix was increased to enhance the mix's stiffness and provide better resistance against permanent deformation. The obtained gradation for the mix was within limits for the 12.5mm mix as per City of Winnipeg Specification CW3410. Figure 3.5 presents the gradation of the blend after adjusting the proportion of Coarse, manufactured fines and fine aggregates.

Table 3.6 Adjusted Blend after Bailey's Method

Sieve Size (mm)	Percent Passing and proportion of blend			Obtained Grading	Specification limits
	Source 1 Coarse (43%)	Source 1 MF (42%)	Source 4 NF (15%)		
19	100.00	100.00	100.00	100.00	100
16	100.00	100.00	100.00	100.00	100
12.5	85.10	100.00	100.00	93.64	90-100
9.5	47.40	99.69	100.00	77.46	75-90
4.75	3.33	79.82	93.84	49.44	48-70
2	0.32	48.85	79.26	32.95	26-45
0.425	0.24	21.28	24.09	13	13-30
0.18	0.21	14.64	2.63	6.61	4-15
0.075	0.16	10.67	0.45	4.59	2-8
Mix Design Method	Superpave				
NMAS (mm)	12.50				
Combined specific gravity	2.73				
Water absorption (%)	0.68				
Aggregate Mineralogy	Basalt				

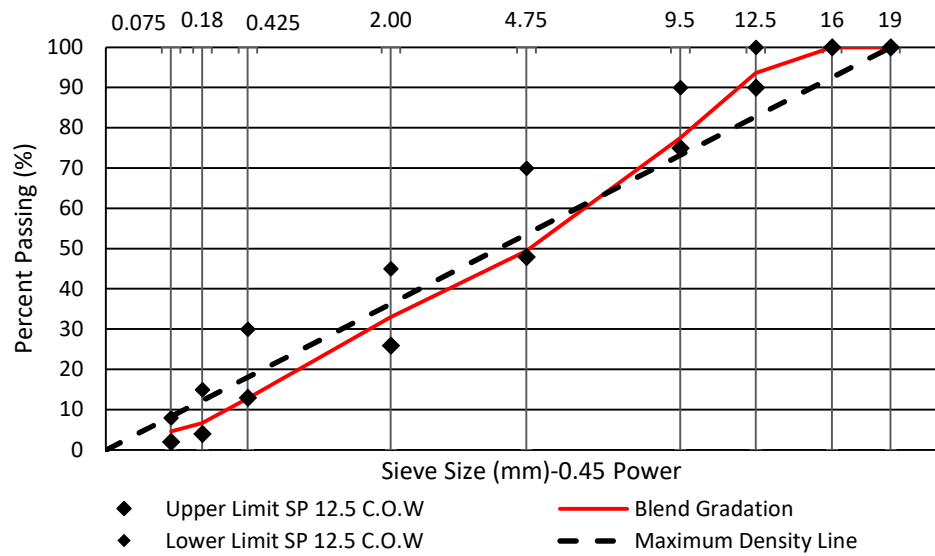


Figure 3.5 Gradation for the Laboratory-Produced Mix

The Bailey method also evaluates the packing behaviour of coarse and fine aggregate fractions through three specific ratios as described by Vavrik et al. (2002):

1. CA Ratio: This ratio focuses on coarse aggregates and reflects the packing efficiency of coarse aggregate particles within the total blend.
2. FA_c Ratio: This ratio indicates the packing characteristics of the coarser fraction within the fine aggregates.
3. FA_f Ratio: This determines how finer portion of the fine aggregates fits together.

Table 3.7 presents the recommended ranges of aggregate ratios for coarse graded 12.5mm NMAs mixtures and an evaluation of the Bailey method for the obtained gradation.

Table 3.7 Bailey Method Evaluation of Laboratory-Produced Mix

Aggregate Ratio	Recommended Range for Coarse Graded 12.5mm NMAS mixtures	Bailey Method Evaluation
CA Ratio	0.50-0.65	0.50
FA _c Ratio	0.35-0.50	0.40
FA _f Ratio	0.35-0.50	0.40

As shown in Table 3.7, the gradation of the laboratory mix satisfies the fine aggregate and coarse aggregate ratio of the mix. The CA ratio meets the lower limit of the recommended range (0.50-0.65) for coarse-graded mixtures. The FA_c ratio is 0.40, which indicates that the coarser fraction of fine aggregates is packing adequately within the blend. FA_f also falls within the recommended range for 12.5mm NMAS mixtures.

3.2.2 Gradation for Plant-produced Mixes

To evaluate the performance of the mix based on the new proposed changes in the specification, five plant-produced mixes were also collected from three projects across Winnipeg. Out of five mixes, four were designed based on the Superpave mix design, while one was developed using the Marshall mix design method. Project 1 SP1 and Project 2 SP2 represent the surface and binder courses, respectively, from one of the City's pilot projects. Similarly, Project 2 SP1 and Project 2 SP2 correspond to the surface and binder courses from another project within the city. The Project 3 MS1 selected for this study was also a surface course mix. All plant-produced mixes utilized aggregates with granite mineralogy and employed Polymer Modified Asphalt (PMA) 64-34P as the binder. The aggregate gradation for each mix is presented in Table 3.8.

Table 3.8 Gradation of Plant-Produced mixes

Sieve Size (mm)	Percent Passing				
	Project 1 SP1	Project 1 SP2	Project 2 SP1	Project 2 SP2	Project 3 MS1
19	100.0	99.5	100.0	100.0	100
16	100.0	97.2	100.0	96.5	100
12.5	95.4	87.5	93.8	86.1	96.1
9.5	85.2	70.0	82.9	75.1	85.2
4.75	49.9	43.5	58.3	50.4	64.0
2.36	36.0	33.3	38.9	32.4	48.9
1.18	30.0	27.3	27	21.9	38.2
0.600	21.2	19.5	17.8	14.7	27.5
0.300	10.2	10.5	9.8	8.6	14.6
0.150	4.6	5.0	5.8	5.4	6.5
0.075	2.8	3.1	4.1	3.9	4.8
Mix Design Method	Superpave	Superpave	Superpave	Superpave	Marshall
NMAS (mm)	12.5	19	12.5	19	12.5
Combined sp. gravity	2.66	2.67	2.62	2.62	2.62
Absorption (%)	0.66	0.55	0.75	0.65	0.94
Aggregate Mineralogy	Granite	Granite	Granite	Granite	Granite
CA Ratio	0.64	0.88	0.80	0.92	0.74
FA _c Ratio	0.58	0.62	0.45	0.43	0.56
FA _f	0.21	0.38	0.32	0.39	0.23

The aggregates for these plant-produced mixes were sourced from different locations than the ones typically used for the City's projects, a strategy replicated from the laboratory-produced mix. Although the locations of these sources were farther away from the city, the superior mineralogy of the aggregates, as compared to limestone, made them a suitable selection for the design of a resistant mix. The CA ratio for SP1 and MS1 mix (12.5mm NMAS) varied due to differences in gradation. Project 1 SP1 CA ratio was within the specified range of 0.50-0.65; however, the Project 2 SP1 mix and Project 3 MS1 mix had considerably higher CA ratios than the specified threshold, which can impact the stability of the mix. Similarly, for 19mm mixes (Project 1 and Project 2 SP2),

both mixes exceeded the desired threshold of 0.60-0.75 for the CA ratio. For FA_C , only Project 2 mixes were within the specified range of (0.35-0.50). The FA_F ratio of mixes was also within the range of 0.35-0.50, except for the Project 1 SP1 and MS1 mix. The Bailey check of gradation reflects that the proportioning of coarse and fine aggregates within the mix was not balanced, with some blends complying with ratios for the coarser fraction and falling short of the finer fractions.

3.2.3 Mix Designs for HMA

For the laboratory-produced mix, a PG 64-34P binder was selected in accordance with the City of Winnipeg's proposed specification CW 3410 R13 for the SP-1 (12.5 mm) mix design, ensuring consistency with the materials used in the City's plant-produced mixes. To prepare the mix, the binder and aggregates were first heated to the designated compaction temperature and blended for two minutes in a mixing bowl as shown in Figure 3.6. The loose mixture was then placed in an oven at the specified temperature and conditioned for two hours. Following this, the mix was transferred into a 150 mm diameter compaction mould and compacted to N_{design} gyrations using the Superpave gyratory compactor, as outlined in AASHTO T312.



(a) Aggregate proportion for Laboratory Mix (b) Mixing aggregates with binder at 155 °C

Figure 3.6 Aggregate Proportions and Mixing for HMA

The Bulk Specific Gravity (G_{mb}) of the compacted specimens was determined according to AASHTO T166 (2022). After compaction with the Superpave gyratory compactor (AASHTO T312), samples were cooled to room temperature and weighed to obtain the dry mass. Each sample was then submerged in a water bath at 25 ± 1 °C for four minutes, and the submerged weight was recorded. To determine the Saturated Surface Dry (SSD) weight, the specimen was removed from the bath, dried with paper towels for 15 seconds, and weighed again.



(a). Conditioning of the Mix for Two Hours at 140 °C (b). Sample After Gyratory Compaction

Figure 3.7 Conditioning and Compaction of the Specimen

The Maximum Theoretical Specific Gravity (G_{mm}) of the mix was measured following AASHTO T209 (2019). For this procedure, binder and aggregates were first heated and mixed for two minutes in the mixing bowl. The loose mix was then conditioned in the oven for two hours before being spread on a tray to cool down to room temperature. Following the cooling of the sample, the mixture was broken down into smaller particles to minimize clumps, the weight was recorded, and then the sample was subjected to a vacuum pressure of 3.7 ± 0.3 kPa for 15 minutes while submerged in water to remove entrapped air. Recording of submerged weight was the final step, after which the maximum theoretical specific gravity (G_{mm}). Figure 3.8 shows the testing setup and procedure for determining G_{mm} and G_{mb} . The values of G_{mb} and G_{mm} were

subsequently used to calculate the volumetric properties of the laboratory-produced mixes

In parallel, plant-produced mixes were also supplied to the laboratory along with their approved mix designs for performance testing. These mixes were reheated to the target compaction temperature (145 °C–148 °C) and compacted using the gyratory compactor to achieve $7 \pm 0.5\%$ air voids, ensuring specimen consistency for testing. The Bulk Specific Gravity (BSG) and Maximum Theoretical Specific Gravity (MTSG) of these plant-produced mixes were calculated using the same procedures. Equations for calculating BSG and MSTG are given below:

$$\text{BSG} = \frac{A}{B - C} \quad (3.1)$$

Where,

A= mass of dry sample (g),

B= Mass of saturated surface dry sample (g),

C= mass of sample submerged in water (g).

$$\text{MTSG} = \frac{A}{A - C} \quad (3.2)$$

Where,

A= mass of oven-dry sample in air (g),

C= mass of sample in water at 25 °C (g)



(a) Loose sample for G_{mm}
(AASHTO T209,2014)



(b) Weight in air of loose
sample



(c) Submerged weight of
loose sample

Figure 3.8 Determination of G_{mm} and G_{mb}

The Volumetric Properties of Plant and laboratory-produced Mixes are presented in Table 3.9.

Table 3.9 Volumetric Properties of Mixes

Volumetric Property	Specified Limits	Lab Mix	Project 1 SP1	Project 1 SP2	Project 2 SP1	Project 2 SP2	Project 3 MS1
OAC (%)	-	5.50	5.00	4.50	4.90	4.55	5.65
VMA (%)	14Minimum	16.12	14.90	13.80	14.10	13.10	15.30
VFA (%)	65-78	66.20	71.30	69.50	71.00	70.20	74.40
P_{be} (%)	-	4.95	4.53	4.11	4.31	4.06	4.93
P_{200}/P_{be}	0.6-1.2	0.92	0.90	1.00	1.00	1.00	-
PG Grade	-	64-34P	64-34P	64-34P	64-34P	64-34P	58-34P
MSTG		2.53	2.49	2.51	2.47	2.47	2.45
BSG		2.42	2.38	2.40	2.37	2.37	2.35
Air Voids		5.9	4.3	4.2	4.1	4.0	3.9

3.3 Test Setup

3.3.1 Moisture Sensitivity Test (AASHTO T283,2022)

To evaluate the moisture sensitivity of the mix, testing was conducted following the AASHTO T283 standard, "Resistance of Compacted Asphalt Mixtures to Moisture-Induced Damage." For the tests, four specimens were produced for each mix with 95 ± 5 mm height and 150 mm diameter.

The loose mixes were first remixed in the mixer at 155°C . After mixing, the blend was allowed to cool at room temperature for two hours. Once cooled, the mix was placed in an oven and cured at 60°C for 16 hours. The cured mix was then compacted using a gyratory compactor set to 60 gyrations. The number of gyrations was determined by compacting multiple samples and adjusting the settings to achieve a target air void content of 7%. The samples were compacted to a height of 95 ± 5 mm, as specified in the test standard. After compaction, the specimens were allowed to cool at room temperature for two hours. Once cooled, the dry weight in air, submerged weight, and weight in saturated surface-dry (SSD) conditions were measured, as shown in Figure 3.9.



(a) Mixing at 155°C



(b) Curing of Mix at 60°C
for 24 hours



(c) Compaction in Gyratory
Compactor

Figure 3.9 Mixing, Curing, and Compaction of the Mix for TSR Specimens

The specimens were divided into two subsets for testing: a wet and a dry subsets with average air voids of 7%. For the wet subset, the specimens were placed in a vacuum container filled with water, and a vacuum pressure of 67 kPa was applied for 10 minutes at room temperature. After vacuum saturation, the specimens were removed from the container and submerged in a water bath for 5 minutes. Following this, each specimen was taken out and lightly dried to determine the saturated surface-dry (SSD) weight, which was used to calculate the degree of saturation. The target degree of saturation for the specimens was 70-80%. The equation for determining the degree of saturation is given as follows:

$$S = \frac{100 J}{V_a} \quad (3.3)$$

Where,

S = Degree of saturation, (%)

J = Volume of absorbed water (cm^3)

V_a = Volume of air voids (cm^3)

Once target saturation was achieved, the saturated specimens were then wrapped in plastic film and placed inside a sealed plastic bag containing 10 ml of water. The bags were stored in an Espec conditioning chamber at -18°C for 16 hours for the freezing cycle, as shown in Figure 3.10.



(a) Saturation in a vacuum container



(b) Specimen conditioning at -18°C for 16 hours

Figure 3.10 Saturation and freezing of the Wet subset

Following completion of the 16-hour freezing cycle, for the thawing cycle, the specimens were transferred to a water bath maintained at 60°C for 24 hours. The plastic bags and films were removed immediately upon placing the specimens in the water bath, and the specimens were conditioned for 24 hours. Following the conditioning period, the wet subset specimens were moved to a 25°C water bath and allowed to equilibrate for two hours before testing, alongside the dry subset as shown in Figure 3.11.



(a) Wet subset in Water Bath at 60°C for 24 hrs (b) Wet subset in water bath at 25 °C for two hrs

Figure 3.11 Conditioning of the Wet Subset in a Water Bath

After conditioning, the thickness of each specimen in the wet and dry subsets was measured using a Vernier calliper. The specimens were then mounted on an Indirect Tensile (IDT) testing machine, where a constant load was applied at a rate of 50 mm/min. The peak load to failure was recorded in the IDT apparatus for both wet and dry specimens. For each subset, tensile strength values were calculated based on Equation 3.4, and then the Tensile Strength Ratio (TSR) was computed based on Equation 3.5.



Figure 3.12 IDT Testing of specimens

The Indirect Tensile Strength (ITS) is calculated as follows:

$$ITS = \frac{2P}{\pi t D} \quad (3.4)$$

Where:

P = Maximum load at failure (N)

t = Specimen thickness (mm)

D = Specimen diameter (mm)

The Tensile Strength Ratio (TSR) is calculated as:

$$TSR = \frac{\text{Average Wet Strength}}{\text{Average Dry Strength}} \times 100 \quad (3.5)$$

3.3.2 Hamburg Wheel Tracking Test

To evaluate the rutting potential of the mixes, the Hamburg Wheel Track Tester (HWTT) was used. HWTT specimens were required to be compacted to 7% air voids at a height of 62.5 mm as per the AASHTO T324 (2023) standard.

For the laboratory-produced mix, aggregates were mixed with the PG 64-34P binder at 155°C and subsequently conditioned in an oven at 140°C for two hours to simulate short-term aging. After aging, the mixture was compacted to a height of 62.5 mm. Once compacted, the specimens were allowed to cool to room temperature, after which their dry, submerged, and saturated surface-dry (SSD) weights were recorded to determine bulk density and air void content.

The specimen preparation procedure for the plant-produced mixes differed slightly from that of the laboratory-produced mix. The loose plant-produced mixes were reheated to the target compaction temperature range of 145°C to 148°C and compacted using a Superpave gyratory compactor to achieve $7 \pm 0.5\%$ air voids, as shown in Figure 3.13. For each mix, four specimens were prepared for HWTT testing.



(a) Mix supplied by the city



(b) Heating the mix to extract HWTT sample



(c) Mix at Compaction Temperature



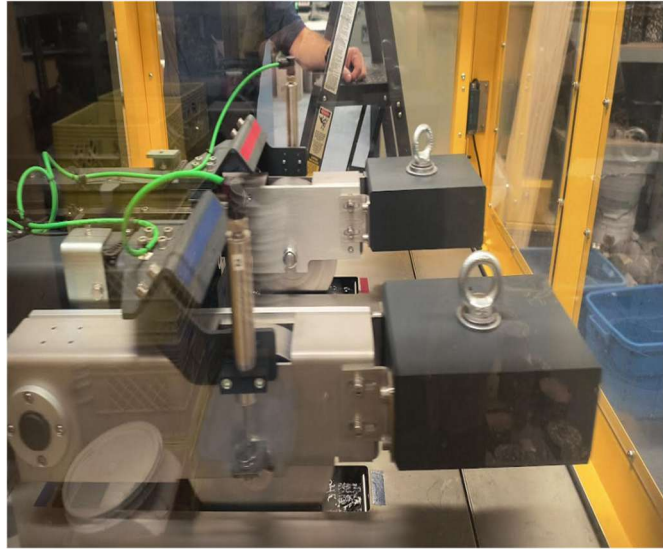
(d) Compacted Samples for HWTT

Figure 3.13 Sample Preparation for HWTT

Compacted samples were then cut using the saw to fit in the HWTT moulds. The moulds were then placed in the Hamburg Wheel Track tester (HWTT) and submerged in water at $46\text{ }^{\circ}\text{C}$ for 45 minutes before starting the test. The specimens were then subjected to 20,000 passes during the test at 52 ± 2 passes/minute with the wheels in HWTT, applying a load of $705 \pm 4.5\text{ KN}$ on the specimen in each pass. The test setup and HWTT device are shown in Figure 3.14.



(a) Specimens in Moulds



(b) HWTT Device

Figure 3.14 Cutting and Placing Specimens in Mould for HWTT

During the test, the specimen deformation is measured along the wheel path through the linear displacement transducers. The output of the test is a plot of rut depth (mm) against the number of cycles, which is referred to as the HWTT curve as shown in Figure 3.15. The curve provides a detailed insight into the deformation behaviour of the asphalt mixture under both mechanical and environmental loading. In the HWTT curve, some key parameters provide insight into rutting resistance and moisture sensitivity of the mixes. The first parameter is post-compaction consolidation, which captures the initial densification of the mix; this occurs during the early stage of loading as the samples reduce air voids and particles are rearranged due to compaction from wheel load. Following the consolidation, the asphalt mix enters a creep phase where the deformation progresses at a constant rate. Two major factors that affect the creep slope are the structural stability of the aggregate skeleton and the stiffness of the binder. Creep slopes generally reflect the rutting potential of the mixes in the absence of significant moisture damage; a higher

value of this slope indicates that the mix is more prone to permanent deformation under load, while a lower value indicates better rutting resistance (Rahman et al., 2014). Meanwhile, the stripping slope focuses on the rate of deformation. Once moisture-induced deterioration becomes the dominant mechanism, the rate of rutting increases during this phase due to loss of adhesion between binder and aggregates, substantially increasing the rate of deformation. A steeper stripping slope indicates higher moisture susceptibility, whereas a milder slope reflects better resistance against stripping (Mohammad et al., 2017). The Stripping Inflection Point (SIP) represents the number of cycles at which the transition from creep to stripping slope occurs. A mix is considered to have greater moisture resistance if the SIP occurs at a higher number of cycles. Rut depth at SIP is also a parameter that corresponds to the Stripping Inflection Point. The maximum rut depth is the representation of total deformation observed in the specimen at the end of 10,000 cycles (Wilhite, 2021). These parameters collectively enable the performance evaluation of asphalt mixes in relation to rutting and moisture susceptibility.

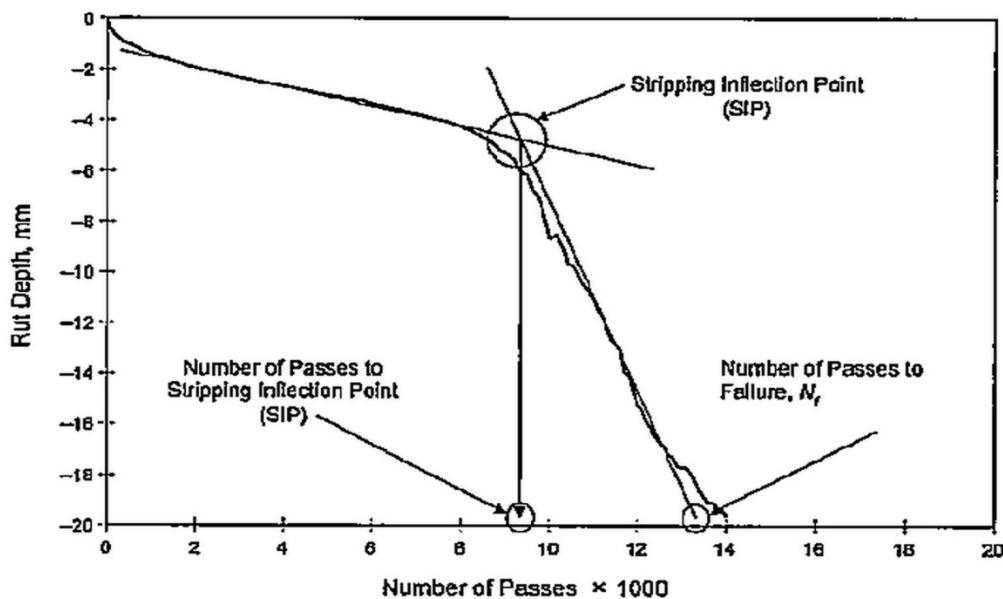


Figure 3.15 Example of HWTT curve (AASHTO T324, 2022)

3.3.3 Dynamic Modulus Test

To evaluate the stiffness characteristics of laboratory produced and plant-produced mixes, the Dynamic Modulus Test (DMT) was used. The test was conducted following the AASHTO T 342 (2022) standard.

The test setup for DMT consisted of a uniaxial hydraulic testing system that applied sinusoidal compressive loading to cylindrical specimens at six loading frequencies: 0.1, 0.5, 1, 5, 10, and 25 Hz. The tests were conducted within an environmental chamber, where the temperature was controlled using a liquid nitrogen system. The specimens were tested at four temperatures, including -10, 4.4, 21.1, and 37.8 °C. Each specimen was positioned between two steel loading plates to ensure uniform load distribution, along with the placement of a thin membrane at the specimen ends to minimize friction.

The laboratory-produced mix was prepared by heating and mixing the aggregate and binder at 155 °C and then conditioning the mix at 140 °C for short-term aging. The aged samples were then compacted in a gyratory compactor with a target height of 170mm at 7% air voids. The loose mixes from different projects were also aged and compacted in the gyratory compactor. 100mm diameter x 150mm height samples were then cored from the compacted samples for DMT. The cored samples were ground 1.0 mm from each end to provide smooth parallel loading surfaces at both ends suitable for deformation measurements.

For the test, three vertical extensometers were mounted on each specimen to measure the axial deformation, positioned at 120 °C around the circumference, with two-gauge studs fixed at each location and a fixed gauge length of 100mm. The test setup and specimen preparation are shown in Figure 3.16.



(a) Test Setup in Chamber



(b) Cored and Grinded Samples

Figure 3.16 Dynamic Modulus Test Setup

The dynamic modulus values for each specimen were determined from average stresses and strains recorded over the final five load cycles at each temperature and frequency combination. The results of all specimens of each mix were then averaged to construct a dynamic modulus master curve following AASHTO R 62 (2013). Plotting mixes on a master curve enables the comparison of mix stiffness across different temperatures and loading frequencies. The master curve was plotted using the sigmoid function from the Mechanistic-Empirical Pavement Design Guide.

$$\text{Log } |E^*| = \delta + \frac{\alpha}{1 + e^{\beta + \gamma \log f_r}} \quad (3.6)$$

Where:

$|E^*|$ = Dynamic Modulus (Psi)

f_r = reduced frequency (Hz)

$\alpha, \beta, \gamma, \delta$ = curve-fitting coefficients

The reduced frequency was calculated using a second-order polynomial time temperature shift factor:

$$\log f_r = \log f + \alpha_1(T_r - T) + \alpha_2(T_r - T)^2 \quad (3.7)$$

Where:

f_r = Reduced frequency at reference temperature (Hz)

f = Loading frequency at test temperature (Hz)

T_r = Reference temperature (°F)

T = Test temperature (°F)

α_1, α_2 =shift factor coefficient

The Excel solver function was used to determine the fitting coefficient ($\alpha, \beta, \gamma, \delta, \alpha_1, \alpha_2$) through numerical optimization using the solver function in MS Excel, which minimized the sum of squared errors between measured and predicted dynamic modulus values. The final master curve was plotted at a reference temperature of 21.1 (70°F).

3.3.4 Asphalt Binder by Ignition Method

To ascertain the binder content of plant-produced mixes, an ignition oven was used for the specimens compacted and tested for the HWTT test. The ignition of specimens for binder content was carried out following the AASHTO T308 (2022) standard.

For the test, the specimen are first heated and oven-dried at 110 °C to soften, break, remove moisture and spread the mix. The loose mix is then divided into two ignition baskets and placed in the furnace, which is already preheated at 538 °C (1000°F). During the test, the furnace records the temperature, mass loss (g) and percentage mass loss at a time interval of one minute. Once the rate of mass loss is consistently 0.01% for three minutes, the test is considered complete. The recovered aggregates after ignition are then removed and cooled to room temperature. The weight of aggregates is recorded to calculate binder content from the difference of weight loss from the initial loose mix. The test setup is shown in Figure 3.17

Since the mixes were plant-produced and not laboratory-constituted, calibration of the ignition oven for aggregate correction factors could not be performed. Therefore, the binder contents reported are uncalibrated values and may include minor errors associated with aggregate mass changes during ignition.



(a) Test Setup in Chamber (b) Ignition Trays with Loose Mix (c) Find Mass of Recovered aggregates (uncalibrated)

Figure 3.17 Ignition Oven Test Setup

3.3.5 Fine Aggregate Angularity Test (AASHTO T304, 2022)

As part of the forensic evaluation of plant-produced mixes, the angularity of the recovered aggregates was also investigated. The evaluation was conducted following the AASHTO T304 standard, which measures the loose, uncompacted void content of fine aggregates. The fine aggregate sample recovered from the ignition oven was first oven dried at 110 ± 5 °C and then allowed to cool at room temperature. The sample was sieved to extract particles retained on a 4.75mm sieve. From passing the 4.75mm sieve, a 190g sub-sample was obtained. The extracted sample was then allowed to flow freely through a standard funnel into a calibrated cylindrical container without tapping or compaction. Excess material was struck off with a glass plate, and the filled container was weighed to obtain the mass of aggregates occupying a known volume. The uncompacted void content was then calculated using the aggregate's bulk dry specific gravity,

from two replicates for each sample. Higher void content values indicated greater particle angularity and surface texture. As per the City of Winnipeg's proposed specification CW3410 R13, the minimum uncompacted void content requirement is 40% for 12.5mm mixes and 45% for 19mm mixes.

Chapter 4. Performance of Laboratory and Plant-produced Mixes

The objective of this study was to analyze and evaluate the rutting potential of mixes after incorporating higher-quality aggregates and polymer-modified asphalt binders. As explained in Chapter 3, the use of Superpave methodology in the mixtures is a departure from traditional Marshall mix designs currently used in City projects. Although the main aim was to evaluate rutting performance, it was also important to reflect the overall performance spectrum of the mix in different conditions and temperatures. Therefore, it was necessary to assess the mixes for moisture sensitivity and their dynamic modulus at various temperatures as well. The first part of this chapter presents the rutting performance of the laboratory-produced and plant-produced mixes, including the rut depths measured at the target number of loading cycles. The subsequent section discusses the results of the moisture sensitivity testing conducted on the same set of mixes, reporting the corresponding dry and wet tensile strength values, as well as the calculated Tensile Strength Ratios (TSR). Additionally, the chapter includes the average dynamic modulus values for each mix and a master curve illustrating the comparative behaviour of the mixes across different temperature ranges and loading frequencies.

4.1 Hamburg Wheel Tracking Test (HWTT) Results

The Hamburg Wheel Tracking Test was performed as per the AASHTO T324 standard. This test assesses the rutting potential and moisture susceptibility of the mixes as samples are subjected to wheel passes under submerged conditions. As part of the experimental program, four samples were prepared and tested for each mix with average air voids of 7%. In total, 24 samples were compacted for six mixes, including one laboratory-produced mix and five plant-produced mixes. The four samples for each mix were divided into two subsets and placed in the molds. The test termination

criteria were (10,000 cycles) 20,000 passes or a maximum rut depth of 14mm, whichever is achieved first. The rut depth recorded over a given number of cycles is shown in Figure 4.1

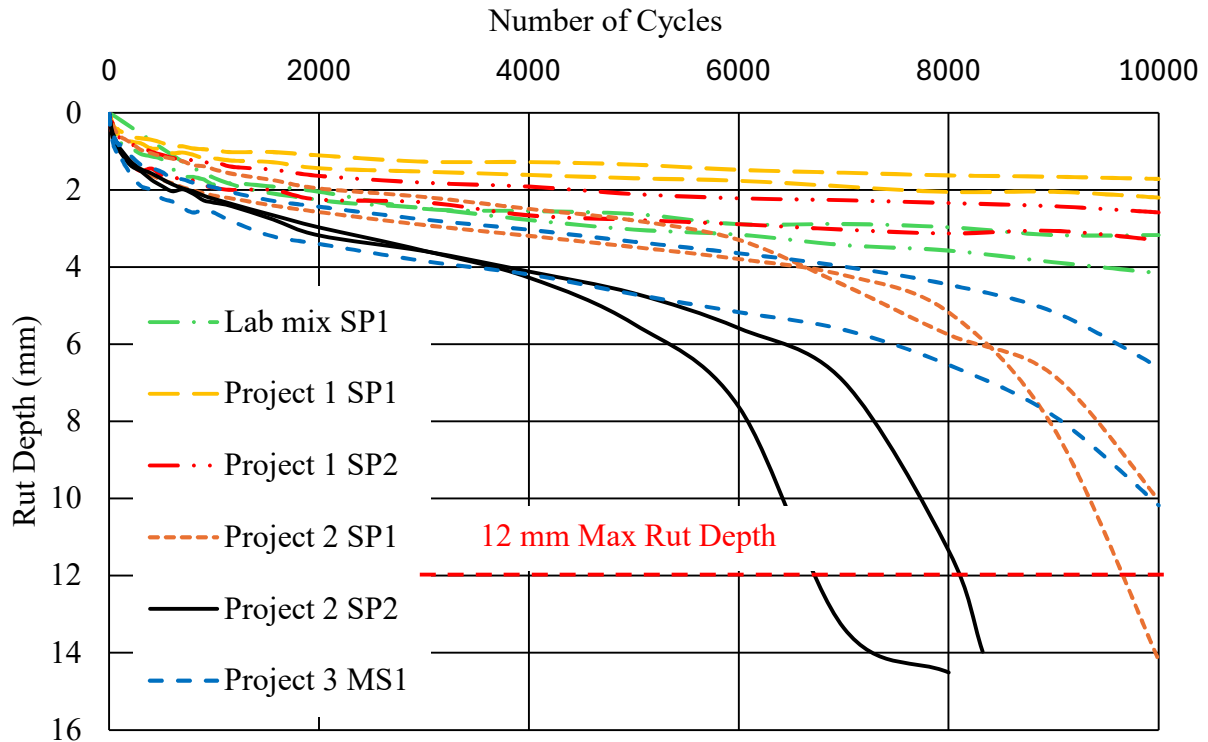


Figure 4.1 HWTT Test Curves for Lab and Plant-produced Mixes.

The mixtures were evaluated based on the creep slope, striping slope, Stripping Inflection Point (SIP), and final rut depth. Lower creep and striping slopes, a higher number of cycles at SIP, and relatively lower rut depth were considered as criteria for superior performance. The average results from four specimens tested for each mix are shown in Table 4.1.

Table 4.1 HWTT Output

Mix	Sample	Creep Slope (micron/cycle)	Stripping slope (mm/cycle)	Number of Cycles to SIP	Rut Depth to SIP (mm)	Max Rut Depth (mm)
Lab Mix SP1	1	0.149	-*	-	-	3.16
	2	0.249	-	-	-	4.15
	Average	0.199	-	-	-	3.65
Project 1 SP1	1	0.106	-	-	-	2.19
	2	0.082	-	-	-	1.71
	Average	0.094	-	-	-	1.95
Project 1 SP2	1	0.147	-	-	-	3.30
	2	0.132	-	-	-	2.58
	Average	0.139	-	-	-	2.94
Project 2 SP1	1	0.331	0.0046	8254	4.51	14.18
	2	0.317	0.0016	6387	3.30	10.04
	Average	0.324	0.0031	7321	3.90	12.11
Project 2 SP2	1	0.766	0.0038	5584	5.37	17.77
	2	0.612	0.0050	7233	6.10	13.97
	Average	0.689	0.0044	6408	5.73	15.87
Project 3 MS1	1	0.361	0.0014	9082	4.60	6.59
	2	0.505	0.0018	8417	6.35	10.17
	Average	0.433	0.0016	8749	5.47	8.38

*Mix did not reach the inflection point

Figure 4.1 and Table 4.1 indicate that the Project 1 SP1 mix outperforms the other mixes under evaluation, exhibiting the lowest creep slope (0.094 micron/cycle) and no stripping, along with the lowest final recorded rut depth of 1.95mm. The HWTT curves remain flat post-compaction, indicating superior rutting resistance and reduced moisture-induced damage. The SP2 mix from the same project (19mm NMA mix) shows slightly higher creep slope (0.139 micron/cycle) and final rut depth of 2.94mm compared to SP1, but still exhibits excellent rutting resistance. The laboratory mix SP1 also shows greater resistance against permanent deformation with a final rut depth of 3.65mm and a low creep slope of 0.199 micron/cycle. The absence of stripping in these

mixes highlights their durability and resistance to moisture damage. The Marshall mix in this study (Project 3 MS1) underwent considerable deformation with the final rut depth of 8.38mm, and the creep slope was moderate at 0.433 micron/cycle. The mix exhibits good moisture resistance with a high number of cycles to SIP at 8749, but deterioration increases once stripping initiates. Project 2 mixes both SP1 and SP2 (surface and binder coarse) did not perform well, HWTT curves exhibited higher creep and stripping slope, with the final rut depth exceeding the cutoff limit to failure (14mm) for SP2 and 12.11mm for SP1. Lab mix SP1, Project 1 (SP1 and SP2) mixes did not undergo any stripping during 10,000 cycles. Although Marshall mixes experienced stripping, a lower stripping slope ensured that the final rut depth was restricted to an average of 8.38mm. In Project 2 SP1, mix stripping was observed at a lower depth as compared to SP2 mix from the same project; however, due to the steeper stripping slope, SP2 mix exceeded the maximum rut depth criteria of 14mm. Comparison of rut depth and depth to SIP is shown in Figure 4.2.

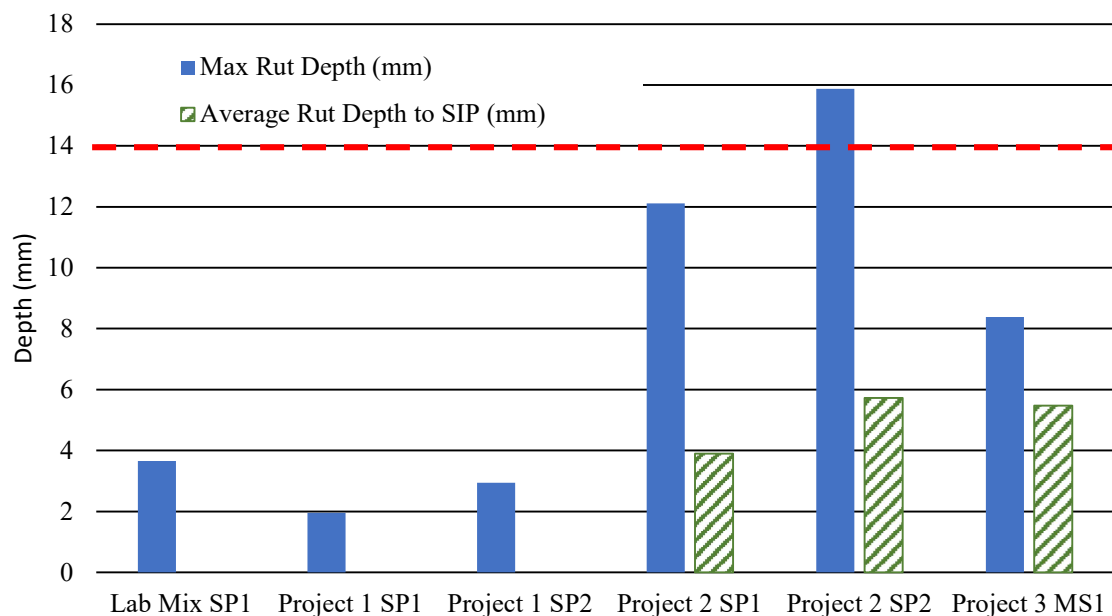
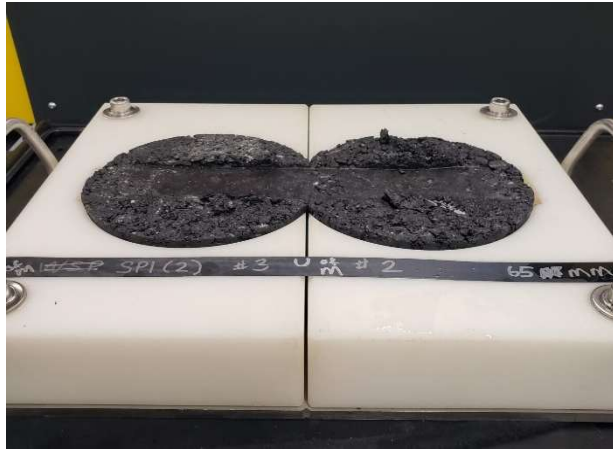
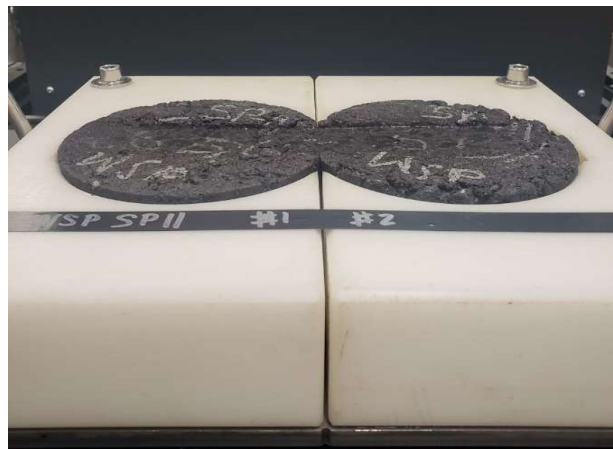


Figure 4.2 Average Rut Depth at SIP and Final Rut Depth Comparison

Among all the mixes evaluated through the Hamburg Wheel Tracking Test, only Project 2 SP2 mix exceeded the rut depth threshold of 14mm. Specimens condition after 10,000 cycles is shown in Figure 4.3.



(a) Lab Mix SP1



(b) Project 1 SP2



(c) Project 1 SP1



(d) Project 2 SP1



(e) Project 2 SP2



(f) Project 3 MS1

Figure 4.3 Rutted Samples After Completion of HWTT.

4.2 Moisture Sensitivity Test Results

Moisture Sensitivity test was performed on different HMA mixes to assess the susceptibility of mixes to moisture-induced damage. For this study, 20 specimens were compacted in the lab to 95 ± 5 mm height and conditioned as described in Chapter 3. The indirect tensile strength for each specimen was recorded in both wet and dry conditions, and a Tensile Strength Ratio was determined for each mix by comparing wet and dry strength. The results for the wet and dry tensile strength ratio for each mix are shown in Table 4.2.

Table 4.2 TSR results

Group	Specimen	Load (N)	Strength (kPa)	TSR (%)
Lab Mix 1	Specimen 1 (Wet 1)	14,320	633.41	80.4
	Specimen 2 (Wet 2)	13,450	596.66	
	Specimen 3 (Dry 1)	16,410	705.49	
	Specimen 4 (Dry 2)	18,850	824.27	
Project 1 SP1	Specimen 1 (Wet 1)	11,450	521.19	56.5
	Specimen 2 (Wet 2)	11,170	495.52	
	Specimen 3 (Dry 1)	19,850	903.55	
	Specimen 4 (Dry 2)	19,270	893.66	
Project 1 SP2	Specimen 1 (Wet 1)	11,410	529.14	68.7
	Specimen 2 (Wet 2)	11,970	553.89	
	Specimen 3 (Dry 1)	18,790	869.47	
	Specimen 4 (Dry 2)	15,260	706.13	
Project 2 SP2	Specimen 1 (Wet 1)	8,410	383.23	69.4
	Specimen 2 (Wet 2)	7,920	360.90	
	Specimen 3 (Dry 1)	12,180	544.36	
	Specimen 4 (Dry 2)	11,780	526.48	
Project 3 MS1	Specimen 1 (Wet 1)	8,340	375.54	81.7
	Specimen 2 (Wet 2)	9,510	440.06	
	Specimen 3 (Dry 1)	11,520	533.06	
	Specimen 4 (Dry 2)	10,330	465.15	

*Project 2 SP1 was not tested due to limited material.

The Laboratory mix exhibited high wet and dry tensile strength, achieving a tensile strength ratio of 80.4% which meets the required minimum tensile strength threshold of 80% outlined in C.O.W specification CW3410. The project 3 MS1 also achieved a TSR of 81.7%; however, the wet and dry strength values were considerably lower than the Laboratory mix. These two mixes had the highest design binder content at 5.5% and 5.65% respectively. These results correlate well with the HWTT findings: the laboratory mix exhibited no evidence of stripping throughout the test, while the Project 3 MS1 mix displayed only a very mild post-SIP stripping slope with a late Stripping Inflection Point, indicating delayed onset of moisture-induced damage. Similarly, the

Project 2 SP2 mix, which exceeded the maximum rut depth criterion in the HWTT, also exhibited a very low TSR, confirming the early SIP and steep stripping slope observed during the wheel tracking test.

Project 1 SP1 and SP2 mixes, despite exhibiting exceptional performance in the HWTT with no stripping observed even after 10,000 cycles, failed to meet the minimum TSR requirement of 80% in the moisture sensitivity test, recording TSR values of 56.5% and 68.7%, respectively. The comparison of wet and dry Tensile strength values for different mixes is shown in Figure 4.4.

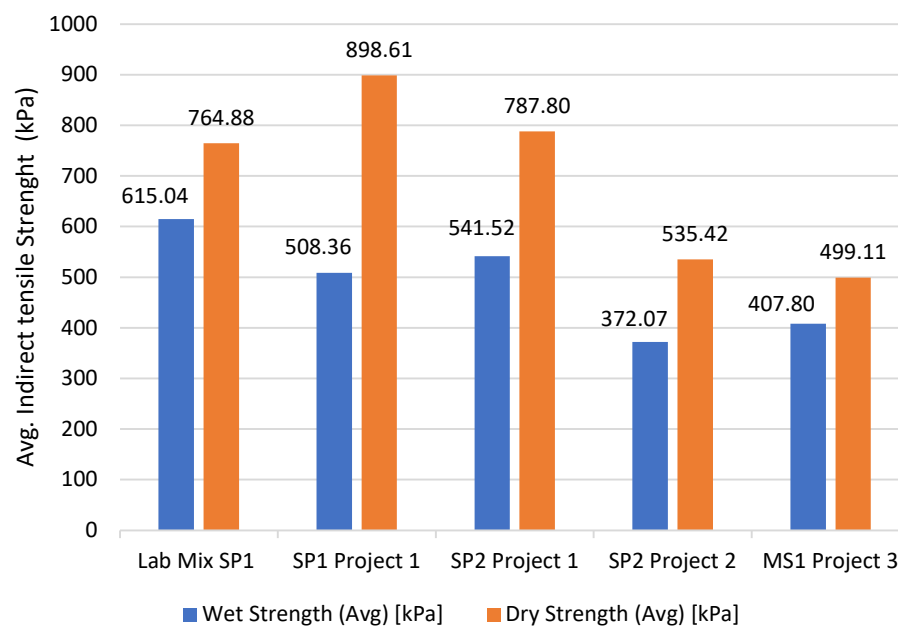


Figure 4.4 Comparison of Wet and Dry Tensile Strength Values

Project 1 SP1 has the highest dry strength values among all mixes; however, the wet tensile strength of 541.42 kPa was significantly lower. A similar trend was observed in the SP2 mix from Project 1 and Project 2 as well, with ITS values markedly different for conditioned and unconditioned specimens.

TSR values of project 1 SP1 mix do not correlate with HWTT results; the mix does show high ITS for the unconditioned specimens; however, the ITS values for conditioned specimens were almost 50% of the dry specimens, which demonstrates a considerable reduction in wet strength of the mix. AASHTO T283 has become one of the most widely used procedures for evaluating the moisture susceptibility of the asphalt mixes; however, despite the popularity of the test, there are some shortcomings as discussed by Varamini et al. (2016). One of the major limitations is that the test does not reliably predict the field performance of the tested mixture, as TSR does not consistently correlate with the stripping resistance of in-service mixtures. Therefore, a mix can have very low TSR values but still exhibit acceptable performance in the field. TSR value is also very sensitive to specimen variables such as air voids, freeze-thaw conditioning temperatures, degree of saturation, and aggregate orientation, which can result in large fluctuations of TSR value if any of these variables change slightly, which reduces the robustness of the prediction of TSR results. Kandhal and Rickards (2001) emphasized that the indirect tensile strength test setup does not replicate the cyclic load that can simulate traffic loading. On the contrary, HWTT provides a more reliable assessment of the stripping potential of the mixes, as specimens during the test are subjected to repeated wheel load in a water bath, and the results correlate well with the field performance. Since both SP1 and SP2 mixes from project 1 showed excellent stripping resistance in the HWTT test, they are expected to demonstrate acceptable performance in the field as well, despite of lower TSR ratio observed in the moisture sensitivity test.

As highlighted by Varamini et al. (2016), the TSR value alone does not provide a reliable indication of the moisture susceptibility of an asphalt mix. Consequently, in addition to the minimum TSR requirement, some transportation agencies specify a minimum wet Indirect Tensile Strength (ITS) in their specifications. While the C.O.W. 3410 specification does not currently

include such a requirement, agencies such as the South Carolina Department of Transportation (SCDOT) and the California Department of Transportation (Caltrans) have adopted this practice. Specifically, SCDOT requires a minimum conditioned ITS of 450 kPa (Karki et al., 2020), whereas Caltrans specifies a minimum conditioned ITS of 482 kPa (San Diego County Department of Public Works). Figure 4.5 compares the measured ITS values of the tested specimens against the minimum thresholds specified by SCDOT and Caltrans.

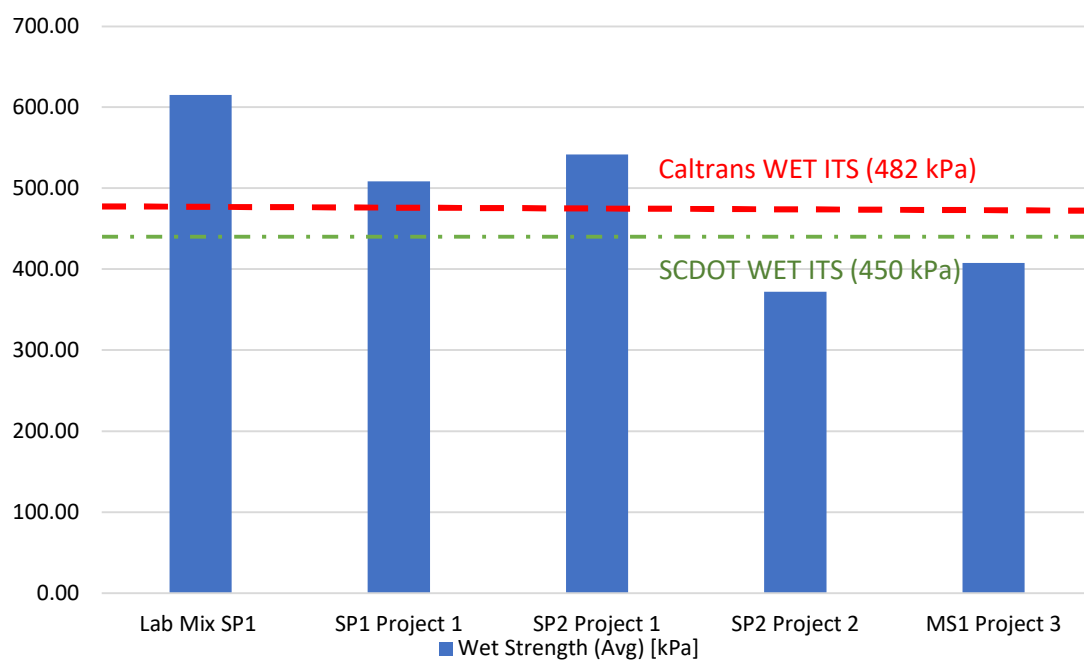


Figure 4.5 Comparison of Wet and Dry Tensile Strength Values

The mixes that did not undergo any stripping in the HWTT test have a higher average wet tensile strength value than the minimum requirement, including the laboratory SP1 mix and Project 1 SP1 and SP2 mixes; however, the Project 2 SP2 mix and Marshall mix MS1 from Project 3 failed to meet the minimum criteria for wet ITS. Although the Project 1 mixes did not meet the required minimum TSR values, their wet ITS values were relatively high, which also confirms well with the HWTT results.

4.3 Dynamic Modulus Test Results

The Dynamic Modulus Test (DMT) was performed following the AASHTO 342 standard (AASHTO, 2022) to evaluate the stiffness characteristics of lab and plant-produced mixes. Prepared specimens were tested under four temperature conditions (-10, 4.4, 21.1, and 37.8 °C) and six loading frequencies (0.1, 0.5, 1.0, 5.0, 10.0 and 25Hz). The average dynamic modulus was then used to construct a master curve following the AASHTO R 62 procedure. (AASHTO, 2013). Figure 4.6 shows the master curves for different mixes. The dynamic modulus values for different temperatures and frequencies are shown in Table 4.3.

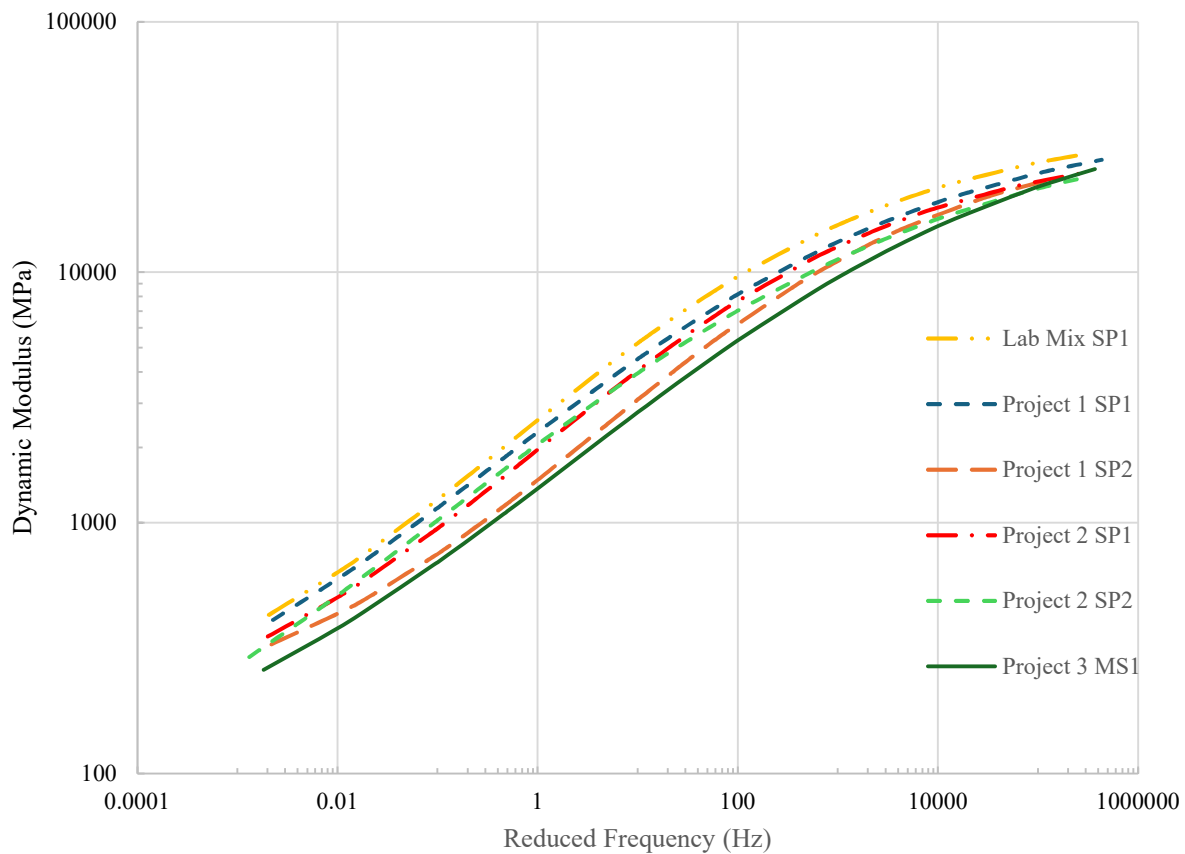


Figure 4.6 Dynamic Modulus Master Curves

Table 4.3 Dynamic Modulus Results

Temperature (°C)	Dynamic Modulus (MPa)					
	Lab Mix SP1					
	0.1 Hz	0.5 Hz	1.0 Hz	5.0 Hz	10 Hz	25 Hz
-10°C	16009.00	20450.00	22401.00	26739.00	28439.00	30408.00
4.4°C	5948.25	8670.70	10117.50	13759.50	15901.50	17892.00
21.1°C	1255.15	1994.80	2447.85	4071.30	5059.00	6345.95
37.8°C	431.18	623.55	747.90	1243.80	1633.05	2299.45
	Project 1 SP1					
	0.1 Hz	0.5 Hz	1.0 Hz	5.0 Hz	10 Hz	25 Hz
-10°C	13561.00	17927.00	19870.00	24550.00	26329.00	28560.00
4.4°C	5044.30	7419.20	8861.50	12600.00	14362.00	16700.00
21.1°C	1096.50	2027.00	2319.80	3436.90	4203.10	5350.70
37.8°C	430.60	582.80	726.30	1164.00	1505.00	2122.60
	Project 1 SP2					
	0.1 Hz	0.5 Hz	1.0 Hz	5.0 Hz	10 Hz	25 Hz
-10°C	9748.10	13394.00	15111.00	19608.00	21308.00	23601.00
4.4°C	2932.60	4624.50	5720.00	8847.00	10419.00	12657.00
21.1°C	798.30	1176.60	1448.40	2459.80	3122.00	4286.10
37.8°C	342.30	430.96	491.12	723.22	921.05	1344.20
	Project 2 SP1					
	0.1 Hz	0.5 Hz	1.0 Hz	5.0 Hz	10 Hz	25 Hz
-10°C	12283.00	16149.00	17685.00	21587.00	23706.00	25693.00
4.4°C	4465.00	6598.30	7787.90	11184.00	12635.00	15086.00
21.1°C	964.10	1506.80	1898.10	3263.30	4010.00	5241.70
37.8°C	368.87	494.18	578.72	911.68	1195.90	1698.60
	Project 2 SP2					
	0.1 Hz	0.5 Hz	1.0 Hz	5.0 Hz	10 Hz	25 Hz
-10°C	13490.00	15235.00	16908.00	21125.00	22924.00	24938.00
4.4°C	4064.40	6092.90	7274.20	9418.20	10860.00	12516.00
21.1°C	1060.70	1651.90	2055.10	3507.50	4215.20	5654.80
37.8°C	318.20	436.31	514.17	819.07	1101.70	1585.00
	Project 3 MS1					
	0.1 Hz	0.5 Hz	1.0 Hz	5.0 Hz	10 Hz	25 Hz
-10°C	10757	14875	16835	21483	23686	26201
4.4°C	3059.1	4578.5	5506.3	8132.4	9424.7	10763
21.1°C	706.29	1094.8	1345.7	2293.2	2866.6	3940.7
37.8°C	275.62	359.8	419.65	634.73	808.47	1217.4

The dynamic modulus $|E^*|$ master curve presented in Figure 4.6 shows the viscoelastic response of six asphalt mixtures evaluated in this study. Lab Mix SP1, Project 1 SP1, Project 1 SP2, Project 2 SP1, Project 2 SP2, and Project 3 MS1 mixes were evaluated across various temperatures and frequencies. The curve follows the typical trend for asphalt mixture behavior as the dynamic modulus increases substantially with loading frequency, corresponding to faster traffic speeds, and decreases at lower frequencies, reflecting the behavior of the mix at slower traffic speeds.

Lab Mix SP1 exhibits the highest stiffness across the reduced frequencies in comparison to other mixes and forms the upper boundary of the master curves. At $-10\text{ }^{\circ}\text{C}$, the $|E^*|$ values of this mix are high, ranging from 16,009 MPa at 0.1Hz to 30,408 MPa, which shows a high risk of thermal cracking at lower temperatures. At high temperature ($37.8\text{ }^{\circ}\text{C}$ at 25 Hz), the mix maintains $|E^*|$ between 431 MPa and 2,299 MPa, indicating excellent resistance to rutting deformation. However, it must be noted that mixes with high stiffness may perform well in rutting, but they are at a higher risk of thermal cracking. Similarly, Project 1 SP1 mix shows high stiffness across different conditions as well, but slightly lower than the Lab mix SP1. At $-10\text{ }^{\circ}\text{C}$, $|E^*|$ values range from 13,561 MPa to 28,650 MPa, reflecting the high rigidity of the mix at low temperatures. In the higher temperature range of $37.8\text{ }^{\circ}\text{C}$, the mix exhibits relatively high stiffness, with values between 430 MPa and 2,122 MPa. Project 1 SP2 exhibits lower stiffness than SP1; however, the $|E^*|$ values at high temperature ($37.8\text{ }^{\circ}\text{C}$) are still relatively high, ranging between 342 MPa to 1,344 MPa. The results of these mixes align with the HWTT results based on high dynamic modulus values observed at high temperatures.

Project 2 SP1 and SP2 mixes exhibit moderate stiffness and consistently plot below Project 1 SP1 and Lab Mix SP1 on the dynamic modulus master curves. However, contrary to expectations based on the HWTT results, where both Project 2 mixes showed significant rutting and permanent

deformation, their dynamic modulus values at higher temperatures were relatively close to those of Project 1 SP1. Project 2 SP1 exhibits dynamic modulus ($|E^*|$) values ranging from 12,283 MPa at 0.1 Hz to 25,693 MPa at 25 Hz and $-10\text{ }^{\circ}\text{C}$, indicating good stiffness at lower temperatures. At $4.4\text{ }^{\circ}\text{C}$, $|E^*|$ drops down, measured between 4,465 MPa and 15,086 MPa, and further decreases under intermediate conditions ($21.1\text{ }^{\circ}\text{C}$), where it varies from 964 MPa to 5,242 MPa. At high pavement temperatures ($37.8\text{ }^{\circ}\text{C}$), the modulus reduces significantly, falling to 369 MPa at 0.1 Hz and 1,699 MPa at 25 Hz, suggesting increased flexibility under warm conditions.

Project 2 SP2 demonstrates slightly better performance than SP1 at intermediate frequencies, but it exhibits lower stiffness under high-temperature, slow-loading conditions. At $-10\text{ }^{\circ}\text{C}$, $|E^*|$ values range from 13,490 MPa at 0.1 Hz to 24,938 MPa at 25 Hz, while at $4.4\text{ }^{\circ}\text{C}$, the range narrows to 4,064 MPa to 12,516 MPa. Under $21.1\text{ }^{\circ}\text{C}$, $|E^*|$ varies between 1,061 MPa and 5,655 MPa, but at $37.8\text{ }^{\circ}\text{C}$, stiffness decreases further to 318 MPa at 0.1 Hz and 1,585 MPa at 25 Hz. These results indicate that while Project 2 SP2 maintains slightly better stiffness than SP1 under moderate conditions, it loses significant rigidity at elevated temperatures.

Project 3 MS1 mix, although it showed good rutting resistance in the Hamburg Wheel Tracking Test, had the lowest dynamic modulus values among all mixes. However, this behavior of the MS1 mix can be attributed to the lower binder grade used in the mix. The performance grade (PG) of an asphalt binder is closely linked to how it behaves under different temperature conditions. In general, binders with higher PG ratings are stiffer, which helps them resist deformation and rutting in hot weather. This added stiffness of the binder directly contributes to a higher overall stiffness of the asphalt mixture. A similar trend was observed by Huang et al. (2008), who reported that the dynamic modulus ($|E|$) * values increased as the binder grade changed from PG 64-22 to PG 70-

22 and then to PG 76-22 at the same temperature, highlighting the strong influence of binder grade on mixture stiffness.

There have been conflicting findings in studies performed on the correlation of dynamic modulus values with rutting performance. Several studies have found a strong correlation between $|E^*|$ and rutting susceptibility of the mixes. Studies conducted by Witczak et al. (2002) and Pellinen and Witczak (2002) found that $|E^*|$ measured at 54.4 °C and 5 Hz is a reliable indicator of the rutting performance of the mixes. In contrast, Birgisson et al. (2004), Ekingen (2004), and Mohammad et al. (2006) reported in their studies that there is little or no correlation when comparing $|E^*|$ values measured at 1-4Hz and 40- 54 °C with rut depth from the Asphalt Pavement Analyzer (APA) test. These inconsistencies in results can be attributed to differences in testing conditions of the Dynamic Modulus Test and the rutting test, such as APA and HWTT. The HWTT setup involves testing specimens under elevated temperature in submerged conditions, which can indicate both rutting susceptibility and moisture sensitivity of the mixes. On the contrary, the DMT test is conducted under controlled dry conditions. The material response evaluated in the two tests is also different, as HWTT focuses on permanent deformation under repeated loading, while DMT assesses the recoverable, viscoelastic strain of the samples.

4.4 Asphalt Binder by Ignition Method

Project 1 and Project 2 mixes exhibited contrasting performance in the HWTT test. While the Project 1 specimens demonstrated superior resistance to permanent deformation under repeated wheel loading, the Project 2 mixes experienced significantly higher deformation. Although both sets of mixes shared similar aggregate mineralogy and followed a comparable mix design methodology, the observed divergence in rutting performance warranted a more detailed investigation to identify the underlying factors contributing to this variation. For this, a forensic

investigation was conducted to determine the asphalt content and aggregate angularity of the tested specimen.

The ignition procedure was carried out as per the AASHTO 308 procedure (AASHTO 308, 2022). The HWTT samples recovered after testing were first heated at 110 °C and then broken down into clumps and smaller fractions. The broken samples were ignited in the oven at 538 °C to burn off the asphalt content and determine the difference in weight for binder content. Fine Aggregate Angularity of the recovered aggregates was also determined using AASHTO T304 (AASHTO T304, 2022). Table 4.4 shows the results from the forensic investigation.

Table 4.4 Asphalt Content and Angularity Values of the Investigated Specimens

Specimen	Optimum Binder Content (%)	AC from ignition oven test (%)	Difference in AC (%)	Uncompacted void content (%)
Project 1 SP1	5.0	5.18	0.18	42.00
Project 1 SP2	4.5	4.22	-0.28	43.23
Project 2 SP1	4.9	4.89	-0.01	39.85
Project 2 SP2	4.5	4.85	0.35	38.53

All the mixes subjected to the ignition showed consistency between Optimum Binder Content (OBC) and actual Asphalt Content (AC) measured through the ignition oven test. Overall deviations were minimal. Project 1 SP1 shows a slightly higher AC than the design, along with the Project 2 SP2 mix. Project 2 SP1 almost shows no significant difference between OBC and AC values. While project 1 SP2 mix had a lower binder than the optimum design binder content.

The angularity values of the recovered aggregates provided valuable insights into the rutting performance observed in the HWTT results. The Project 1 mixes exhibited consistently higher angularity values, exceeding the 40% minimum threshold specified by the City of Winnipeg (C.O.W) CW 3410 specification, which contributed to their superior resistance to permanent

deformation under repeated wheel loading. In contrast, the Project 2 mixes demonstrated slightly lower angularity values, falling just below the specified lower limit of 40%, which likely reduced aggregate interlock and shear resistance, resulting in the higher deformation observed during HWTT testing.

Chapter 5. Analysis of Mix Performance

A summary of the mixes evaluated in this study, along with their key mix design parameters, is presented in Table 5.1. These mixes represent a range of material properties typically observed in municipal asphalt mixtures. Table 5.2 summarizes the ranking of different mixes across different performance tests as observed during this study.

Table 5.1 Key mix properties

Mixes	%AC	FAA (%)	PG Grade	Aggregate Type	Mix Design Method
Lab Mix SP1 (SP12.5)	5.5	45.00	64-34P	Basalt	Superpave
Project 1 SP1 (SP12.5)	5.0	42.00	64-34P	Granite	Superpave
Project 1 SP2 (SP19.0)	4.5	43.23	64-34P	Granite	Superpave
Project 2 SP1 (SP12.5)	4.9	39.85	64-34P	Granite	Superpave
Project 2 SP2 (SP19.0)	4.5	38.53	64-34P	Granite	Superpave
Project 3 MS1	5.6	43.60	58-34P	Granite	Marshall

Table 5.2 Ranking of mixes across different performance test

Mixes	Max Rut Depth (mm)	HWTT Rank	TSR (%)	TSR Rank	Wet ITS (kPa)	Rank	Dynamic modulus @ 37.8 and 0.1Hz	DMT Rank
Lab Mix SP1	3.65	3	80.4	2	615.04	1	431.1	1
Project 1 SP1	1.95	1	56.5	5	508.36	3	430.6	2
Project 1 SP2	2.94	2	68.7	4	541.42	2	342.3	4
Project 2 SP1	12.11	5	-*	-	-	-	368.8	3
Project 2 SP2	15.87	6	69.4	3	372.07	5	318.2	5
Project 3 MS1	8.38	4	81.7	1	407..80	4	275.6	6

* Project 2 SP1 was not tested for TSR

The laboratory-produced Superpave mix (SP1) was developed using high-quality basalt aggregates, which exhibited superior engineering properties. The fine aggregates demonstrated high angularity (49.19%), ensuring improved friction and resistance to deformation, while the

coarse aggregates had the lowest Los Angeles (LA) abrasion loss value of 14.1%, indicating excellent durability. To further enhance performance, a polymer-modified binder (PG 64-34P) was incorporated, offering greater stiffness at elevated service temperatures and improved low-temperature cracking resistance.

This optimized combination of aggregate structure and binder selection resulted in outstanding performance across all evaluation criteria. In the Hamburg Wheel Tracking Test (HWTT), the mix recorded only 3.65 mm rut depth after 10,000 cycles (20,000 passes), with no evidence of stripping throughout the test duration. Similarly, the Dynamic Modulus ($|E|$) * values for Laboratory mix SP1 were the highest among all evaluated mixes, signifying superior stiffness and a reduced tendency toward permanent deformation. The Tensile Strength Ratio (TSR) was also close to the 80% threshold, thereby meeting the minimum requirement for moisture resistance and confirming the mix's resilience against water-induced damage.

A key design takeaway was the deliberate adjustment of the gradation curve, wherein the proportion of fines was reduced and the coarse fraction increased to establish a denser aggregate skeleton with strong stone-on-stone contact. While the Laboratory mix adopted a denser gradation, the Voids in Mineral Aggregate (VMA) remained higher than that of several plant-produced mixes. Since VMA governs the space available for binder, this characteristic ensured sufficient effective asphalt content. Unlike mixes with lower VMA, which often compromise binder film thickness and durability, Laboratory SP1 was designed with a slightly higher binder content than the Plant produced mixes, thereby optimizing rutting resistance without sacrificing durability.

This balance between a dense aggregate skeleton and adequate binder film thickness was reflected in the moisture sensitivity and rutting performance outcomes. Whereas plant-produced mixes exhibited stripping and marginal TSR values, Lab SP1 demonstrated excellent adhesion and no

stripping damage under HWTT, underscoring the success of its optimized gradation and binder content. The cross-sectional gradation distribution of SP1 is illustrated in Figure 5.1.

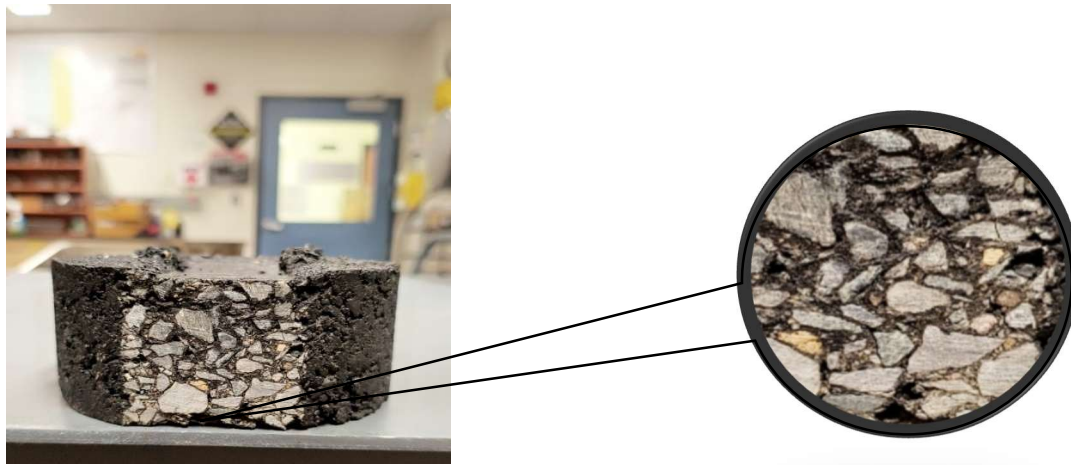


Figure 5.1 Aggregate skeleton of Lab Mix SP1

Figure 5.1 shows the coarse aggregate skeleton of the mix, with voids between coarse aggregates filled with manufactured fines and natural fine aggregates. Similarly, the higher binder content in the mix ensured that the aggregates were held together when high stress was applied to the surface during the HWTT test.

In contrast, the Project 2 SP1 and SP2 mixes, which exhibited the lowest rutting resistance during the HWTT test, also had the lowest average binder content and the lowest average VMA values among all the Superpave mixes. While lower VMA typically indicates a denser aggregate skeleton with tightly packed particles and reduced void space, this condition must be complemented by relatively higher binder content. Otherwise, the resulting inadequate binder film thickness can lead to mechanical degradation of the mix under repeated loading during the HWTT test. This effect was evident in the severely rutted samples from Project 2, as illustrated in Figure 5.2.

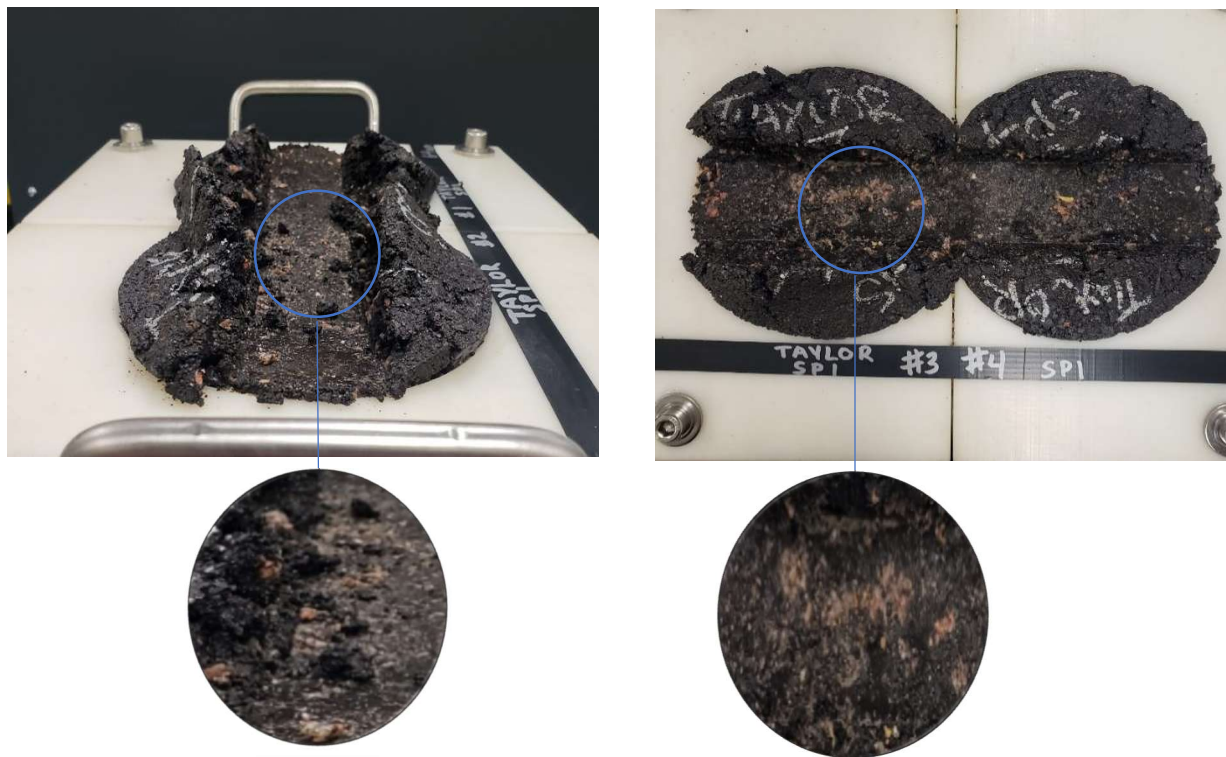


Figure 5.2 Uncoated Aggregates Observed in Rutted Specimens of Project 2

In addition to the lower VMA and reduced effective binder content, the Project 2 mixes also exhibited lower uncompacted void content values, as revealed by the forensic investigation. These combined factors contributed to the excessive deformation observed in the Project 2 mixes during the performance testing.

To better understand the impact of volumetric properties on the rutting performance of the tested mixes, a correlation analysis was conducted on various parameters, including VMA, VFA, Asphalt content (AC), and effective binder content (P_{be}). The correlation values represent the correlation coefficient, which measures the strength and direction of the linear relationship between two variables. The coefficient ranges from -1 to +1. A negative R value indicates an inverse relationship, which implies that as one variable increases, the other variable decreases. The

magnitude of the R value (positive or negative) also has a significant bearing on the strength of correlation; if it's closer to 1, the correlation between two values is stronger. Table 5.3 shows the correlation between volumetric properties, rut depth, TSR ratio, Wet Strength and Dynamic modulus values at 0.1 Hz and 37.8 °C

Table 5.3 Correlation of Rut Depth, DMT, and TSR values with volumetric properties

Performance Measure	Volumetric Property			
	AC	P _{be}	VMA	VFA
Rut Depth	-0.25	-0.39	-0.59	-0.27
TSR	0.62	0.57	0.44	-0.06
Wet Strength	0.72	0.75	0.84	-0.66
DMT @ 0.1 Hz and 37.8°C	0.05	0.17	0.42	-0.65

As discussed in Chapter 2, Voids in Mineral Aggregate (VMA) are strongly associated with the rutting performance of asphalt mixtures. Several studies have emphasized the importance of maintaining adequate VMA, noting that lower VMA values may create a dense aggregate skeleton but fail to provide sufficient space for asphalt binder. When low VMA is combined with reduced asphalt content, the result is often inadequate binder coating on the aggregates. Conversely, such mixes tend to exhibit higher Voids Filled with Asphalt (VFA) values due to the reduced air voids present.

The present findings show that VMA values exhibit a moderate negative correlation with rut depth measured in the Hamburg Wheel Tracking Test (HWTT), indicating that lower VMA levels contribute to greater rutting. This trend is consistent with observations in the literature. Similarly, effective binder content also demonstrated a moderate negative correlation with rut depth, suggesting that as effective binder content increases, rutting decreases. This highlights the role of optimal binder film thickness in coating aggregates. Mixes with excessively thick binder films

may still fail rutting tests, as aggregates are prone to sliding under heavy loads, while inadequate film thickness can also compromise rutting resistance.

Asphalt content (AC) and effective binder content (P_{be}) exhibited a moderately strong correlation with the Wet Strength values. Both parameters showed a positive relationship with the mixtures' resistance to moisture-induced damage, aligning well with trends reported in the literature. An optimal binder layer thickness coating the aggregates is important for improved moisture damage resistance. When the layer is too thin, it fails to cover the aggregate's surfaces adequately, leaving the areas exposed to moisture and making the mixture more vulnerable to stripping and adhesion loss. Table 5.3 shows a relation of 0.72 and 0.75, respectively, to the Wet Strength, which indicates binder may decrease the moisture susceptibility of the mixes; however, care must be taken while increasing the binder content, as excessively thick binder film can lead to binder-rich zones, where aggregates are less interlocked and more prone to sliding under repeated loading. Similarly, there is a high positive correlation of Wet Strength values with VMA, which suggests that sufficient void space for binder film thickness may enhance binder aggregate bonding and limit stripping. Lab mix SP1 mix had the highest VMA among the mixes evaluated in this study at 16.12%, which resulted in the highest wet strength value of 615.04 kPa. Conversely, mixes with low VMA, such as the Project 2 SP1 mix, which had a VMA value of 13.1% only, lower than the 14% minimum limit specified in the city specifications, had a lower Wet Strength value of approximately 372.07 kPa. Mixes with higher AC and P_{be} showed relatively higher TSR values, such as Laboratory mix SP1 and Project 3 MS1 with AC of 5.5% and 5.65% respectively, but the correlation of Asphalt content with wet strength values was not consistent, with some mixes with higher binder content exhibiting lower wet strength. The correlation of TSR values with VFA was weak; some mixes had higher VFA than others, but failed to demonstrate acceptable TSR performance, which goes

to show that higher VFA does not necessarily translate to better coating or thicker films. A mix with lower air void content can have higher VFA values, and still fail in moisture damage, as it is governed by film thickness or adhesion, and not just the percentage of air voids filled.

The Dynamic Modulus Test (DMT) and Hamburg Wheel Tracking Test (HWTT) did not show a strong correlation; some of the mixes that demonstrated higher permanent deformation in the HWTT test showed higher stiffness in the DMT test, such as project 2 mixes. Table 5.4 shows the correlation of Dynamic Modulus values with rut depth and creep slope.

Table 5.4 Correlation Between DM Values and HWTT Output Parameters at 37.8°C

Parameter	Frequency					
	0.1Hz	0.5Hz	1.0Hz	5.0Hz	10Hz	25Hz
Rut Depth	-0.55	-0.46	-0.47	-0.42	-0.36	-0.37
Creep Slope	-0.67	-0.55	-0.54	-0.48	-0.42	-0.42

From Table 5.4, it can be observed that the rut depth and creep slope exhibited the strongest correlations with the dynamic modulus at 0.1 Hz, compared to other loading frequencies. Both parameters showed a moderate negative correlation with DM values, indicating that as the dynamic modulus decreases, rut depth and creep slope tend to increase. This relationship is logical, as a lower dynamic modulus reflects reduced stiffness and greater susceptibility of the asphalt mixture to permanent deformation under repeated loading.

Among the evaluated mixtures, the Laboratory Mix SP1 demonstrated the highest stiffness across all mixes in the DMT test, with a value of 431 MPa at 0.1 Hz and 37.8°C. This high stiffness is consistent with its superior rutting performance in the HWTT test, where it exhibited a low deformation of 3.65 mm. The alignment of these two independent performance tests reinforces the

understanding that mixtures with greater stiffness at low frequencies (speed) are generally more resistant to rutting.

However, there were notable exceptions, particularly with the Project 2 mixes. These mixes, despite showing higher stiffness in the DMT test, also recorded higher rut depths in the HWTT test. This apparent inconsistency can be explained by recognizing that dynamic modulus primarily characterizes the mixture's linear viscoelastic stiffness under controlled sinusoidal loading, whereas the HWTT subjects the mixture to repeated rolling loads under water at elevated temperatures, which introduces additional damage mechanisms. For example, a mix may exhibit high stiffness due to a dense aggregate structure or a stiffer binder, but if the aggregate gradation lacks sufficient stone-on-stone contact or if the binder-aggregate bond is weak, the mixture can still deform under traffic loading.

Moreover, moisture susceptibility plays a critical role in HWTT performance but is not captured in the DMT. Mixtures with higher VMA may allow greater binder film thickness and higher voids, which, while contributing to stiffness in the DMT, can also create pathways for water infiltration in HWTT, accelerating stripping and permanent deformation. Similarly, higher VFA values can reduce air voids to the extent that moisture cannot escape, under repeated wheel loading

Therefore, the discrepancy between DMT and HWTT results highlights the influence of non-linear damage mechanisms (plastic flow, stripping, and aggregate rearrangement) that are not represented in modulus measurements. This reinforces that while stiffness is a valuable predictor of rutting resistance, it must be interpreted alongside tests that capture moisture effects, aggregate interlock, and viscoelastic flow, to provide a complete picture of mixture performance.

Figure 5.3 illustrates the correlation between Dynamic Modulus ($|E|$) and HWTT rut depth at 20,000 passes and 46°C. The correlation was slightly stronger with an R^2 value of 0.31 but still considered weak. This indicates that while $|E^*|$ provides some insight into rutting resistance, the testing conditions of HWTT (submerged specimens, rolling steel wheel loading, and elevated temperature) capture additional mechanisms of permanent deformation not reflected in Dynamic Modulus testing. This highlights that laboratory stiffness measurements ($|E^*|$) alone cannot fully characterize rutting resistance under simulated field moisture and loading conditions.

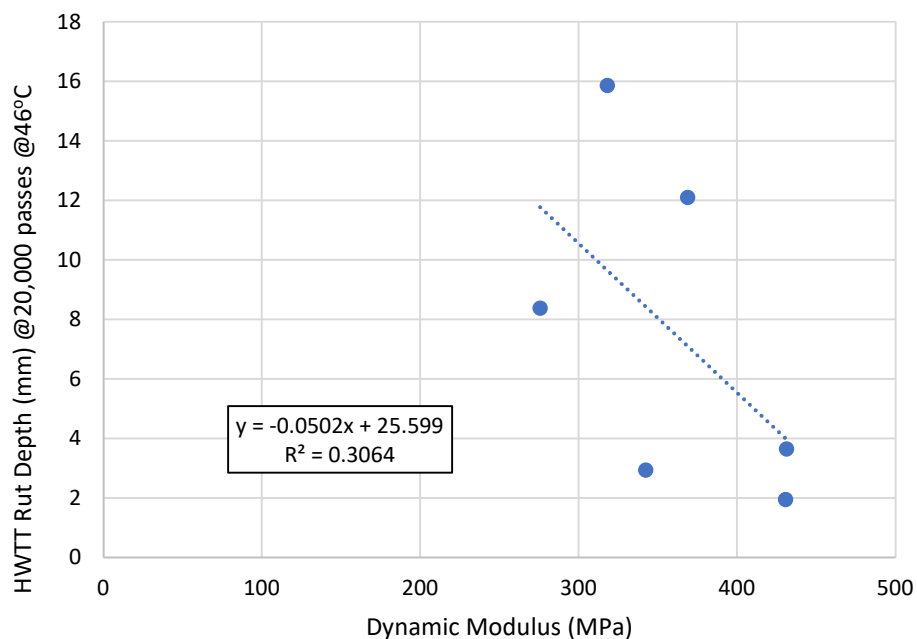


Figure 5.3 Dynamic Modulus $|E^*|$ at 37.8°C at 0.1 Hz vs Rut Depth at 20,000 passes

Figure 5.4 shows the correlation between Dynamic Modulus ($|E|$) and Wet Strength values. The linear regression yielded an R^2 value of 0.37, indicating a weak relationship. This suggests that TSR (a moisture sensitivity indicator) is not strongly explained by $|E^*|$ at the tested condition. The weak correlation can be attributed to the fact that Wet Strength is primarily influenced by

aggregate–binder adhesion and moisture damage resistance, rather than stiffness or viscoelastic properties, as captured by $|E^*|$. Therefore, Dynamic Modulus alone may not be a good predictor of moisture susceptibility.

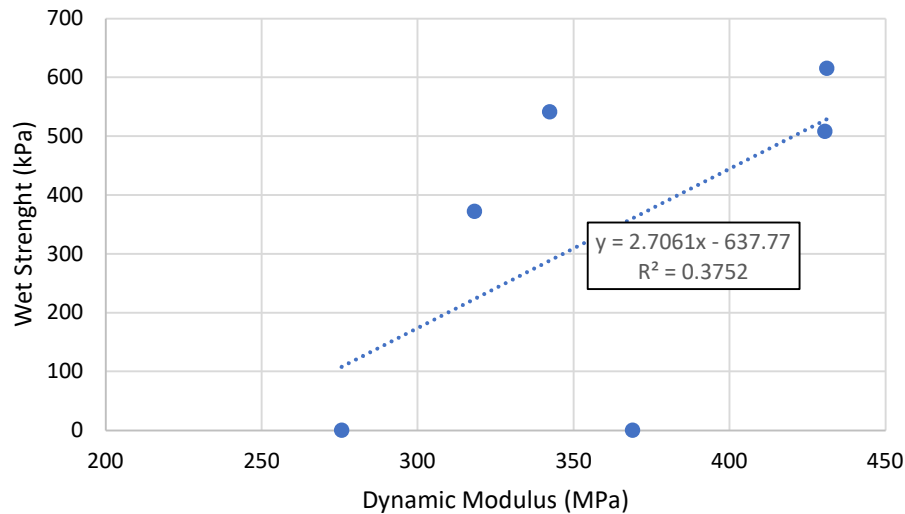


Figure 5.4 Wet Strength vs Dynamic Modulus $|E^*|$ at 37.8°C at 0.1 Hz

Figure 5.5 illustrates the relationship between HWTT rut depth and Wet Strength values. The regression analysis yielded an R^2 value of 0.0492, indicating that the two parameters are only weakly related. This suggests that Wet Strength, which indicates mixture's integrity in wet conditions, is not strongly influenced by rut depth. The weak link can be explained by the fact that each parameter reflects different mechanisms: Wet Strength depends on binder–aggregate adhesion and resistance to stripping, while HWTT rut depth reflects permanent deformation from repeated wheel loading, influenced further by factors such as temperature and water submersion during testing.

In practical terms, this means a mixture could perform well in rutting resistance but still show poor Wet Strength, or vice versa. For instance, a stiff mix with a strong aggregate structure may resist

deformation under HWTT loading but fail to maintain adequate adhesion for moisture-induced damage. Conversely, a mix with good binder coverage may pass Wet Strength requirements yet remain vulnerable to rutting under traffic. This shows that HWTT rut depth alone cannot predict TSR performance, and both tests are needed to depict a complete picture of mixture durability.

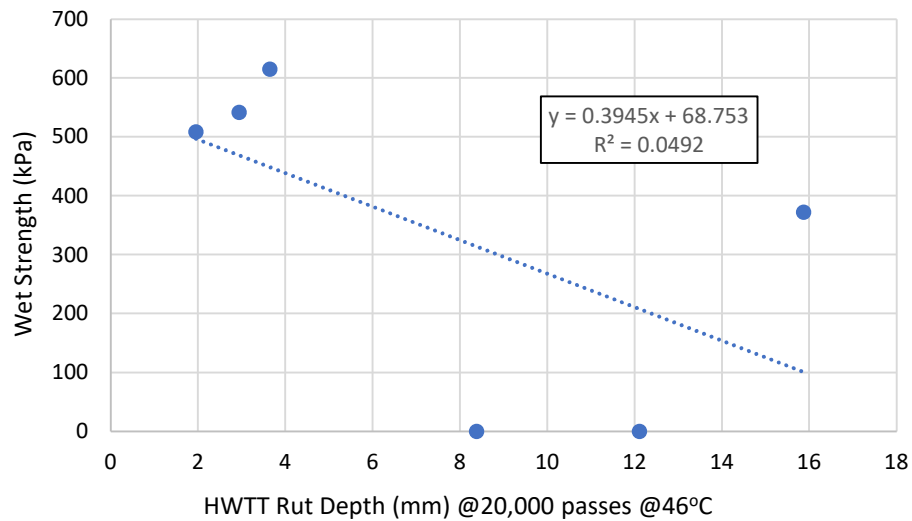


Figure 5.5 Wet Strength vs Rut Depth at 20,000 passes

Chapter 6. Conclusions and Recommendations

This study aimed to design and evaluate asphalt mixtures for heavy-duty pavements in Winnipeg, where traffic loading and harsh climatic conditions place higher demands on rutting resistance and mix durability. The study placed greater emphasis on investigating alternative aggregate sources and employing aggregates with harder mineralogy, such as Basalt and Granite, to improve resistance to permanent deformation. Six mixes were evaluated in this study, out of which one was a laboratory-produced Superpave mix, four were plant-produced Superpave mixes, and one was a plant-produced Marshall mix. These mixes were tested for their rutting resistance, moisture susceptibility, and elastic modulus using the Hamburg Wheel Tracking Test (HWTT), Tensile for moisture-induced damage, and Dynamic Modulus Test (DMT). These tests provided a comprehensive assessment of how binder quality, aggregate gradation, and volumetric design parameters affect field performance.

6.1 Conclusions

Based on the analysis, the following conclusions can be made from the research:

1. The HWTT test for all the mixes showed higher rut resistance except the Project 2 mixes (SP12.5 and SP19), which exhibited considerably higher deformation, and the Project 2 SP2 mix exceeded the maximum rut depth limit (14mm at 20,000 passes) during the test. Lower VMA and lower effective binder content were associated with poor rutting performance.
2. Moderate negative correlations were observed between rut depth and both VMA and effective binder content (P_{be}), with a coefficient correlation of -0.59 and -0.39, respectively. This reflects the importance of maintaining adequate VMA and binder

content in the mix for better rutting performance; the balance between aggregate skeleton and binder content is essential. An overly dense aggregate skeleton with insufficient voids or binder content compromises durability and rut resistance.

3. The tensile strength ratio of these mixes exhibited variable results, with only the Laboratory SP1 mix (SP12.5) and Project 3 (Type IA) mix meeting the minimum 80% TSR requirement as per the specification. Project 1 and Project 2 mixes exhibited lower TSR; however, Project 1 mixes met the minimum ITS value requirement despite a lower TSR ratio observed during the test.
4. TSR values showed a moderately positive correlation with Asphalt Content (AC), effective binder content (Pbe), and VMA, with correlation coefficients of 0.62, 0.57, and 0.44, respectively. This indicates that when designing high-stiffness, rut-resistant mixes, it is important to balance aggregate structure with binder content, ensuring adequate binder film thickness for proper aggregate coating while maintaining sufficient VMA to provide space for effective binder distribution.
5. The correlation between HWTT and Wet Strength was weak ($R^2 = 0.0492$), indicating that moisture sensitivity tests alone are not sufficient to predict the long-term stripping potential of asphalt mixtures. A mixture may exhibit a low Wet Strength value yet still demonstrate excellent rutting and moisture resistance in the HWTT, underscoring the importance of using performance-based tests, such as the HWTT, alongside conventional binder-aggregate tests for more reliable mix evaluation.
6. The Dynamic Modulus exhibited weak correlations with both Wet Strength ($R^2 = 0.37$) and HWTT ($R^2 = 0.30$), likely due to the differences in testing mechanisms and

conditions between these tests. This indicates that stiffness alone does not fully capture a mixture's moisture susceptibility or rutting performance.

6.2 Recommendations

1. The study revealed that designing rut-resistant mixes for asphalt pavements can be realistically achieved by combining hard, angular aggregates with controlled gradation, polymer-modified asphalt, and adequate volumetric properties (VMA and effective binder content). These improvements in the specification and mix design practices are achievable since they build on materials, methods, and testing protocols already adopted by other Canadian municipalities, such as Edmonton and Toronto. These steps, combined with performance testing, can be incrementally implemented within Winnipeg's mix design and quality control framework for developing rut-resistant mixes.
2. The laboratory-produced mix exhibited significantly higher stiffness across all tested temperatures in the DMT and showed low rut depths as well. The primary difference between the laboratory-produced and plant-produced mixes was Bailey's check of gradation. Notably, several cities, such as the City of Edmonton, have already mandated the use of the Bailey Method in their specifications. Incorporating the Bailey Method into the specifications could further enhance the rutting performance of City mixes in the future.
3. The inclusion of volumetric property limits in the specification is a positive transitional step toward achieving better mix designs. However, the observation that some mixes exhibited high rut depths despite meeting the volumetric criteria highlights the need to

incorporate performance-based requirements. Introducing performance tests, similar to practices adopted by major municipalities such as the City of Toronto and the City of Edmonton, would ensure more reliable and durable pavement performance.

4. The research focused on evaluating the rutting performance and moisture susceptibility of the mixes using the TSR test. However, considering Winnipeg's cold climate and frequent freeze-thaw cycles, it is recommended to include low-temperature cracking tests to better assess the performance of the mixes under extreme conditions.
5. It is recommended to evaluate the field performance of different pilot projects across the city to gain better insights into their long-term durability and behaviour under actual traffic and environmental conditions. Such evaluations would provide valuable data to guide the City of Winnipeg in making informed decisions for optimizing future pavement design, material selection, and project planning.

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