IMPLEMENTATION OF LABORATORY CONDITIONING AND TESTING PROTOCOL TO EVALUATE MOISTURE SUSCEPTIBILITY OF ASPHALT MIXTURES

Final Report

PROJECT SPR 835

RESEARCH

Oregon Department of Transportation

IMPLEMENTATION OF LABORATORY CONDITIONING AND TESTING PROTOCOL TO EVALUATE MOISTURE SUSCEPTIBILITY OF ASPHALT MIXTURES

Final Report

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16. Abstract: Moisture damage in asphalt mixtures can cause early cracking and rutting failures due to the internal damage accumulated by the high internal pore pressures created at the aggregate-binder interface and/or within the binder phase by heavy traffic loads. Due to the high precipitation levels and frequent rain events, distresses originating from moisture damage are commonly observed on roadways in Oregon. ODOT has been mostly using hydrated lime to combat distresses related to moisture damage at the mixture level, while the effectiveness of new chemical anti-strips and warm-mix technologies has also started to be investigated. However, a reliable moisture conditioning method and moisture susceptibility test need to be developed and implemented for Oregon to determine the possible long-term impact of several new additive technologies on pavement longevity. Roadway geometry, asphalt layer density, construction of proper superelevation on the roadway for effective water removal, and functioning drainage facilities can be considered to be the other important factors that control moisture-related failures on roadways. Based on the comprehensive literature review and the results of the laboratory investigations, this study recommends the use of a colorimeter in conjunction with the current AASHTO T 283 (2014) method to determine the adhesion and cohesion-related moisture susceptibility. According to the laboratory test results, vacuum saturation is able to create significant moisture damage in the asphalt microstructure, and no other conditioning method needs to be adapted to replace the vacuum saturation method. Developed tools and test procedures are expected to help ODOT identify the benefits of recent additive technologies that are being developed to combat moisture damage of asphalt mixtures.

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SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in^2	square inches	645.2	millimeters squared	mm^2
ft^2	square feet	0.093	meters squared	m^2
yd^2	square yards	0.836	meters squared	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	kilometers squared	km^2
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	ml
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	meters cubed	m^3
yd^3	cubic yards	0.765	meters cubed	m^3
NOTE: Volumes greater than 1000 L shall be shown in m^3.				
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams	Mg
		TEMPERATURE (exact)		
°F	Fahrenheit	(F-32)/1.8	Celsius	°C

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm^2	millimeters squared	0.0016	square inches	in^2
m^2	meters squared	10.764	square feet	ft^2
m^2	meters squared	1.196	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	kilometers squared	0.386	square miles	mi^2
		VOLUME		
ml	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	meters cubed	35.315	cubic feet	ft^3
m^3	meters cubed	1.308	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.205	pounds	lb
Mg	megagrams	1.102	short tons (2000 lb)	T
		TEMPERATURE (exact)		
°C	Celsius	1.8C+32	Fahrenheit	°F

*SI is the symbol for the International System of Measurement

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DISCLAIMER

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1.0 INTRODUCTION

Moisture damage in asphalt mixtures can cause early cracking and rutting failures due to the internal damage accumulated by the high internal pore pressures created at the aggregate-binder interface and/or within the binder phase by heavy traffic loads. The loss of adhesion and cohesion are the two major mechanisms causing moisture damage in asphalt mixtures. Loss of adhesion occurs when water seeps between binder and aggregate, physically dislocating binder from aggregate; loss of cohesion occurs when the binder strength is weakened in the presence of water, again due to high pore pressures created by vehicular loads. These mechanisms work in tandem to strip binder from aggregate and damage the binder phase in the asphalt concrete mixture, thereby accelerating pavement degradation. Adhesion between the aggregate and the binder is generally expected to be the major factor controlling the moisture sensitivity of the asphalt mixture. Due to the high precipitation levels and frequent rain events, distresses originating from moisture damage are commonly observed on roadways in Oregon. ODOT has been mostly using hydrated lime to combat distresses related to moisture damage at the mixture level, while the effectiveness of new chemical anti-strips and warm-mix technologies has also started to be investigated. However, a reliable moisture conditioning method and moisture susceptibility test need to be developed and implemented for Oregon to determine the possible long-term impact of several new additive technologies on pavement longevity. Roadway geometry, asphalt layer density, construction of proper superelevation on the roadway for effective water removal, and functioning drainage facilities can be considered to be the other important factors that control moisture-related failures on roadways.

In this study, the most reliable and promising asphalt mixture conditioning and testing methods to evaluate moisture susceptibility are determined. Developed tools and test procedures are expected to help ODOT identify the benefits of recent additive technologies that are being developed to combat moisture damage of asphalt mixtures. A recent survey conducted by National Center for Asphalt Technology (NCAT) (West 2018) revealed that the majority of the contractors and state DOTs in the U.S. suggest the use of the Hamburg Wheel Tracking Test (HWTT) as a moisture susceptibility test. Repeated loading of the asphalt specimen while submerged in water is expected to simulate the field moisture damage more effectively than the modified Lottman method [with tensile strength ratio (TSR)] that is currently being used by ODOT. However, several research studies in the literature provided results that do not completely support the use of HWTT for moisture susceptibility evaluation with the current AASHTO T 324 (2019b) specification (Izzo and Tahmoressi 1999; Lu 2005; Tsai et al. 2016). Results of the extensive testing with the HWTD conducted in Europe also suggested that the test should only be used for rutting evaluation, as it has a minimal indication of moisture susceptibility (European Committee for Standardization (ECS), 2020 - ‘‘Small size device”, Procedure B conducted in the air; Tsai et al. 2016). Since the HWTT test provides results that are affected by both rutting resistance and moisture susceptibility, not being able to isolate the rutting performance-related component of the measured response from the total rutting may result in false conclusions regarding the moisture resistance of the asphalt mixtures. For this reason, the effectiveness of HWTT and other candidate moisture susceptibility tests in identifying the moisture resistance of Oregon asphalt mixes needs to be determined. Using the most reliable test and conditioning method to quantify moisture resistance is expected to help ODOT select the most effective strategies and additive technologies to combat moisture-related early failures.

1.1 ORGANIZATION OF THIS RESEARCH REPORT

This research report is structured as follows:

- Chapter 2.0 provides a comprehensive literature review summarizing moisture damage mechanisms, factors controlling moisture damage, and testing and conditioning methods for moisture susceptibility evaluation.

- Chapter 3.0 summarizes the asphalt mixture plant sampling process and the general mixture properties.

- Chapter 4.0 discusses the test and conditioning methods used in this study for moisture susceptibility quantification with comprehensive data analysis methods. Results from the conducted tests and major conclusions are also discussed in this section.

- Chapter 5.0 summarizes all major conclusions, provides suggestions for implementation, and recommends future work.

1.2 KEY OBJECTIVES OF THIS STUDY

The main objectives of this study are to:

- perform a detailed evaluation of several different tests and methods for moisture susceptibility quantification of asphalt mixtures;

- determine the most effective moisture susceptibility test and conditioning methods that can identify the impact of chemical anti-stripping agents, warm-mix additives, and lime on stripping resistance;

- develop and recommend a detailed test procedure for the selected experiment;

- using the developed test protocol, determine the impact of different anti-stripping agents and warm-mix additives on moisture susceptibility of Oregon asphalt mixtures; and

- determine and report the effectiveness of recent technologies for in-situ infiltration and porosity measurements.

2.0 LITERATURE REVIEW

In this comprehensive literature review, the mechanisms behind asphalt moisture damage accumulation, moisture conditioning methods, and testing procedures were explored and evaluated by examining existing literature with the intent to identify improvements to ODOT's existing moisture susceptibility testing and conditioning methodologies.

2.1 MOISTURE DAMAGE MECHANISM

Asphalt pavement moisture damage results from the failure of the monolithic microstructure of the asphalt mixture due to the pore pressures created by the infiltrated water and the applied vehicular loads. This damage type is commonly referred to as stripping. The loss of adhesion and cohesion are the two major mechanisms causing moisture damage in asphalt mixtures (see Figure 2.1). Loss of adhesion occurs when water seeps between binder and aggregate, physically dislocating binder from aggregate; loss of cohesion occurs when the binder strength is weakened in the presence of water, again due to high pore pressures created by vehicular loads. These mechanisms work in tandem to strip binder from aggregate and damage the binder phase in the asphalt concrete mixture, thereby accelerating pavement degradation (Hicks 1991). Mixture level stripping has been shown to be influenced by aggregate mineralogy and chemistry, aggregate surface texture, asphalt binder chemistry and the chemical and electrical interaction between the aggregate and binder (Bonaquist et al. 2007). Wet environments accelerate stripping damage and, when combined with the high internal pore pressures generated by vehicle traffic, accelerate asphalt pavement degradation. Pavement distress from stripping severely reduces the designed lifespan of asphalt concrete and manifests as raveling, potholes, fatigue cracking, rutting, or bleeding.

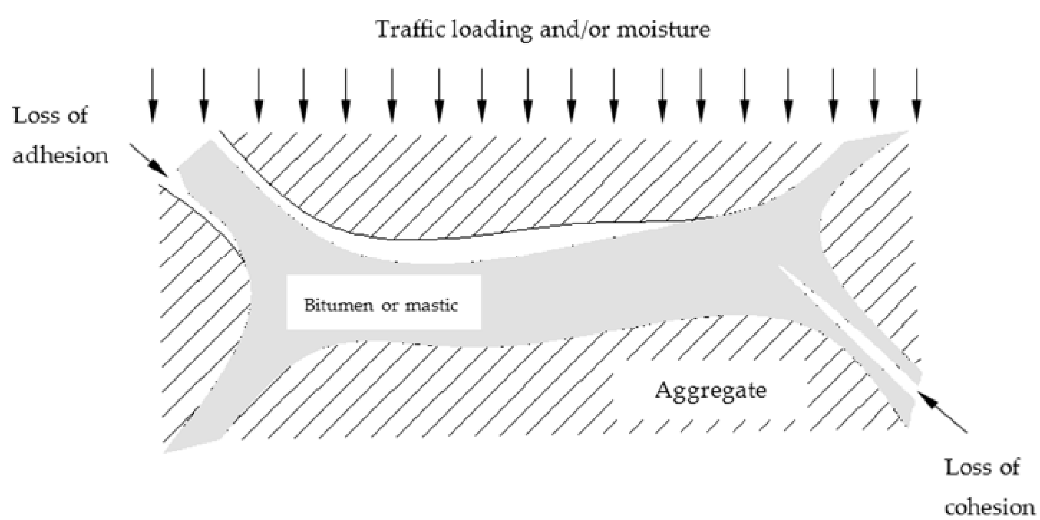


Figure 2.1: Loss of adhesion and cohesion of asphalt pavement (Diab et al. 2017)

Although the loss of adhesion and cohesion are the two major mechanisms causing moisture damage in asphalt mixtures, the delamination caused by water infiltrating through the pavement and damaging the bond between the two pavement layers is also a common mechanism for moisture-related pavement failure (Khosla et al. 1999; Scholz and Rajendran 2009). For this reason, bonding between the pavement layers, density and permeability of the asphalt layers, roadway geometry, and the presence and functionality of drainage facilities along the roadway are also other factors controlling the moisture susceptibility of asphalt surfaced pavements. It is possible to have a roadway constructed with the least moisture susceptible asphalt mixture and still end up with severe stripping issues due to roadway design and construction mistakes. For these reasons, it is not possible to eliminate moisture damage by just improving asphalt mixtures' moisture resistance. However, improving the mixture quality to reduce stripping is an important factor in combating moisture-related failures on roadways. In this study, only the asphalt mixture level moisture susceptibility was evaluated to determine the most effective and practical conditioning and testing method for Oregon.

2.2 FACTORS CONTROLLING MOISTURE DAMAGE AND STRATEGIES TO REDUCE MOISTURE SUSCEPTIBILITY

2.2.1 Asphalt Binder Content and Properties

As one of the key constituents in asphalt concrete mixtures, asphalt binder content and associated properties can drastically impact a given mix design's moisture susceptibility and durability. Properties of asphalt binder that can control the moisture susceptibility of the asphalt mixtures include: i) film thickness, ii) viscosity, iii) chemical composition, iv) resistance to aging, and v) surface energy (Diab et al. 2017).

Because of the high variability of binder properties used for asphalt mixture production today, the Superpave Performance Grade System (Superpave PG) was developed by the Strategic Highway Research Program to classify the properties of a binder according to AASHTO M 320 (2017b), AASHTO T 313 (2019a), AASHTO T 314 (2016), and AASHTO T 315 (2020) specifications. Binder viscoelastic properties are highly influenced by temperature; therefore, binders are evaluated and graded for their low-temperature cracking resistance, high-temperature rutting resistance, and intermediate-temperature fatigue cracking resistance (Gibson et al. 2012). A Superpave PG rating is expressed as PG XX-XX, with the first number representing the 7-day max temperature for a climate region in ºC and the second number denoting the minimum pavement temperature in ºC. Environmental conditions, traffic density, and vehicle load duration are essential considerations in choosing the appropriate binder grade for asphalt mixture production.

Several researchers in the literature hypothesized that asphalt binders with higher viscosity levels (stiffer binder types) could resist moisture-related stresses more than the softer binders (Hicks 1991; Graf 1986). Graf (1986) indicated that for two asphalt mixtures with similar binder viscosity levels, binder chemistry's impact on moisture susceptibility is negligible. However, Petersen et al. (1974 and 1982) indicated that certain forms of carboxylic acids and sulfoxide compounds in asphalt binders could be highly affected by moisture and result in cohesion-related failures in some asphalt mixtures.

Figure 2.2 shows that thick binder films exhibit lower cohesive tensile strength while thin binder films exhibit lower adhesive tensile strength. However, mastic thickness can vary in asphalt pavements, leading to failure from either loss of cohesion or loss of adhesion, though one type usually dominates the failure mechanism of a particular mixture design (Lytton et al. 2005). It is generally observed that loss of adhesion is the major mechanism resulting in moisture-related asphalt mixture failures.

A survey conducted by Hicks (1991) showed that none of the state D.O.T.s participated in the survey (including Arizona, California, and Oregon D.O.T.s) think that asphalt binder has a significant impact on the moisture susceptibility of asphalt mixtures, while the aggregate characteristics were indicated to be the most important factor. However, the impact of asphalt binder properties on cracking resistance, compactibility, and density were not evaluated in the survey. Only the cohesion and adhesion properties were investigated.

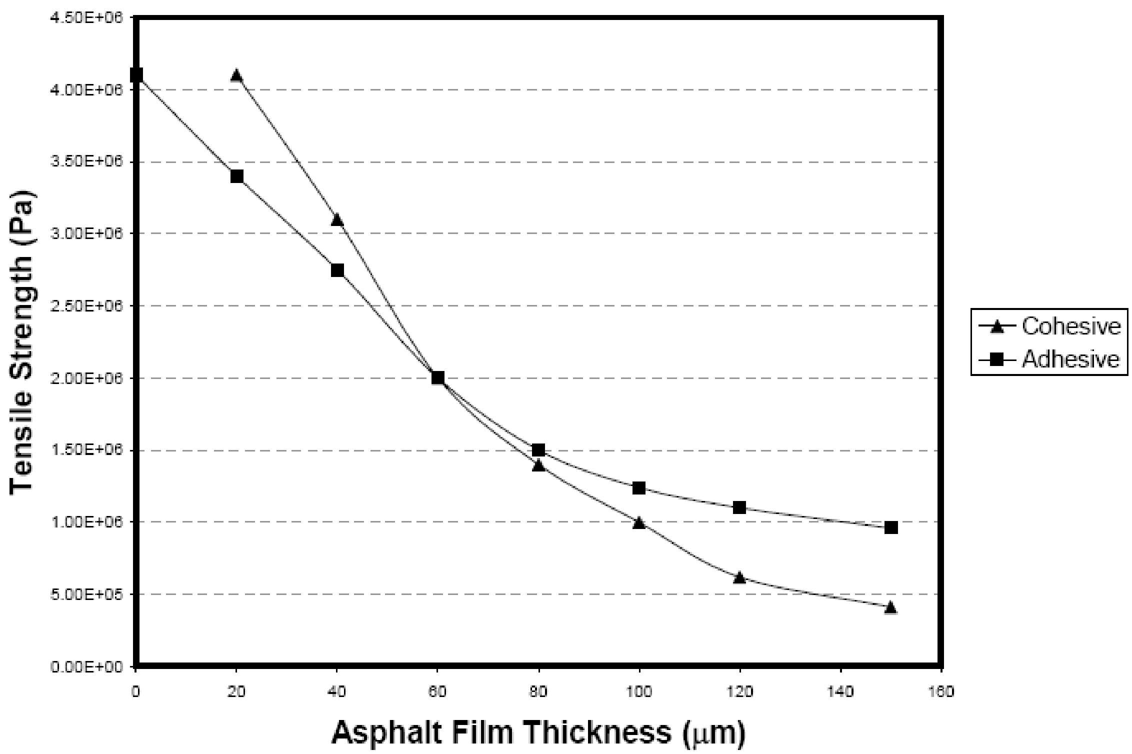


Figure 2.2: Tensile strength versus film thickness (Lytton et al. 2005)

Asphalt binder content composes only 4% to 8% by weight of the total mixture (Bonaquist et al. 2007), yet accounts for 30% to 40% of the total cost of the constructed pavement structure. For this reason, it is essential to thoroughly optimize mixture design and maintain high tolerance during mixture preparation; even a 0.2% increase in binder content can be costly. Though cost is important during mix preparation, it is also essential to achieve the desired in-place air void content and ensure durable pavements. Too much binder in a mixture might over compact to an excessively low air void content, bleed during or after construction, or experience rutting, while too little binder might result in high air void content and high permeability (due to compaction issues during construction), increased moisture susceptibility, and reduced strength (Lytton et al. 2005).

2.2.2 Aggregate Properties

Aggregates compose approximately 92% to 96% of the total volume of HMA, which makes aggregate properties an essential consideration for mixture design and additive selection. Aggregates can degrade at various rates under different conditions depending on mineralogy. Aggregate dust content is important, as too much dust can inhibit uniform binder coating, thereby increasing moisture susceptibility. Aggregate surface texture also plays a significant role in stripping resistance. For example, a smooth surface is stripped of binder more easily than a rough surface, the latter of which strengthens the mechanical adhesion of the binder (Scholz and Rajendran 2009). Hydrophobic aggregates tend to be more resistant to stripping, while hydrophilic aggregates tend to resist stripping less as the water more readily displaces the binder (Scholz and Rajendran 2009). Research has shown that aggregate pH plays a role in asphalt concrete moisture susceptibility due to its ability to change the pH of water that comes in contact with the aggregate surface and accelerate or retard the stripping of the binder from the aggregate (Howson et al. 2011).

Though aggregate selection has a significant impact on binder adhesion (therefore moisture susceptibility) within asphalt mixtures, it is often impractical to transport large volumes of aggregate over long distances. For this reason, most state highway agencies produce asphalt with a blend of locally available aggregates. For instance, the Oregon Department of Transportation (ODOT) produces asphalt concrete with widely available crushed river rock mixed with fine sand. This results in a wide array of mineral and chemical compositions, which can be difficult to match with a binder - a common problem when using locally available aggregates (Scholz and Rajendran 2009).

Many state agencies simply avoid sources of aggregates known to exhibit stripping or augment the mixture with anti-stripping additives. ODOT commonly uses lime to coat aggregates prior to mixing to encourage binder adhesion (Scholz and Rajendran 2009), but the large storage volume required and questionable health impacts of fine lime dust particles incentivize the exploration of alternative anti-stripping additives.

The mechanisms behind some aggregate qualities are discussed in the following sections.

2.2.2.1 Shape, Texture, Angularity, and Porosity

Mechanical adhesion between the asphalt binder and aggregate is generally controlled by the surface texture of the aggregates (also called the “surface roughness”). Rougher aggregate surface with high texture and pores can absorb asphalt binder and provide a stronger mechanical bond between the aggregates and the binder. Based on the results of forensic investigations conducted on asphalt concrete samples taken from field sections, it was indicated by McBain and Hopkins (1929) that retention of asphalt binder on glassy-surfaced aggregates is significantly less than the binder retention on aggregates with high surface texture due to the weaker mechanical bond between the aggregate and the binder. However, other studies in the literature (Hicks 1991) also showed that high aggregate surface texture and porosity could often result in lower asphalt film thickness around the aggregates, which ultimately reduces the bonding between the aggregates and binder. For this reason, aggregate shape, texture, angularity, and porosity need to be carefully evaluated when designing asphalt mixtures. Aggregates with higher texture and porosity would require increased binder content to achieve similar film thicknesses to their less porous counterparts. The Superpave mixture design method has different control experiments to account for these factors when determining the asphalt binder content.

On the other hand, when the aggregate pores are very small, capillary action allows water to travel easily through them once the aggregate surface is exposed directly to water. This mechanism grants water access to a larger area of the binder-aggregate interface through only a small exposed section of an aggregate surface (Cho and Kim 2010). To delay this mechanism, it is important that the binder uniformly coats as much of the aggregate surface as possible during the mixing process. Careful selection of binder, aggregate, and mixture additives can help ensure uniform binder coating and strong adhesion, thereby improving the stripping resistance of asphalt concrete.

Although higher aggregate angularity may increase the rutting resistance of asphalt concrete mixtures, poor coating due to higher surface aggregate surface area and limited binder amount in the mixture can result in lower adhesion between the binder and the aggregates. In addition, research showed that sharp aggregate edges could rupture the asphalt binder film around the highly angular aggregates and result in an asphalt mixture with poor moisture resistance (Hicks 1991). Water can easily seep through the broken asphalt binder film and fill the zone between the binder and the aggregate. With the application of heavy vehicular loads, high water pressures around this zone can separate the aggregates from the binder and result in severe stripping.

2.2.2.2 Surface Chemistry

Aggregate surface chemistry can be challenging to study due to the wide variety of aggregate mineralogy and the interaction of each to a specific binder. However, research has shown that aggregate surface chemistry contributes both to initial binder adhesion and long-term stripping resistance. To optimize binder-aggregate coating and adhesion, it is best to combine acidic aggregates with basic binders, or basic aggregates with acidic binders (Scholz and Rajendran 2009).

Research has shown that binder-aggregate adhesion weakens in the presence of higher pH water and could contribute to stripping. For example, Howson et al. (2011) indicated that increasing water pH from 7.0 to 9.0 reduced binder-aggregate adhesion strength. However, it was also determined that the change in water pH resulting from contact with the aggregate surface in HMA was minimal and exhibited little effect on the adhesive strength of the aggregate-binder interface. However, other environmental factors that raise the pH of infiltrated water could accelerate stripping and, therefore, pavement degradation.

Hydrophilic aggregates generally attract water particles, which results in the accumulation of thin water films around the aggregates. With heavy vehicular traffic, this thin water film can create high levels of water pressure, which can result in the loss of bonding between the aggregate and the binder. Hydrophilic aggregates are generally acidic aggregates with high silica contents, while hydrophobic aggregates are generally basic with low silica contents. As expected, hydrophobic aggregates, such as limestone, have better moisture resistance than hydrophilic aggregates (Majidzadeh et al. 1968). However, it should be noted that locally available aggregates are generally used for asphalt mixture production, and it is generally not cost-effective (or environmentally friendly) to haul aggregates for long distances. For this reason, coating the locally available aggregates with lime powder is often a preferred method to increase the hydrophobic properties of locally available aggregates. It was also indicated that the presence of iron, magnesium, calcium, and aluminum on the aggregates could improve the bonding between the asphalt binder and the aggregates, while sodium and potassium on the aggregate surface may reduce the overall moisture resistance (Hicks 1991).

2.2.2.3 Electrical Properties

Aggregate surface electrical charge plays a significant role in the adhesive and cohesive bond strength between binder and aggregate. By selecting a binder and aggregate with a predominantly opposite electrical charge, a stronger aggregate-binder bond is possible, thereby increasing stripping resistance and pavement longevity (Scholz and Rajendran 2009). Aggregate surface charge is largely governed by the mineralogy of the aggregate, of which silica and alkaline earth oxides have the largest impact. Increased silica content correlates to a more negative surface charge, while increased alkaline earth oxides correlate to a more positive surface charge (Hicks 1991). It is possible to influence aggregate or binder surface charge with a mineral or chemical additive to optimize bond strength and reduce stripping. Though an aggregate stockpile may exhibit a predominant surface charge, it is important to note that aggregate surface charge will always exhibit both positive and negative qualities due to the various mineral compounds contained.

2.2.2.4 Cleanliness

The cleanliness of the aggregates is another important factor controlling the moisture resistance of asphalt mixtures. Excessive clay dust on the aggregate surface cuts the bonding between the asphalt binder and the aggregate, resulting in adhesion issues and ultimately stripping. The aggregate surface also needs to be available to share electrons with the asphalt binder without any blockage to be able to form better bonding. For these reasons, the aggregates' clay content must be measured and controlled by the sand equivalent test (ASTM D2419, 2022c and AASHTO T 176, 2017a) by following the requirements of the Superpave mixture design method.

2.2.3 Lime

As stated in the previous section, it is often impossible to haul limestone for long distances to improve the moisture resistance of asphalt mixtures. Using locally available aggregates is almost always more economical and creates less environmental damage (Cass and Mukherjee 2011).

For this reason, coating other aggregate types with hydrated lime to improve the hydrophobic properties of the aggregates is an effective method of reducing the moisture susceptibility of asphalt mixtures.

Adding lime into an HMA mixture has been shown to increase stripping resistance, even when used with aggregates susceptible to stripping (Trejo et al. 2014). For this reason, lime is widely used to strengthen HMA mixtures and is mixed with the aggregate prior to adding the asphalt binder. The addition of lime instills a positive charge to the aggregate surface and, when used with a negatively charged binder, can improve aggregate-binder bond strength and binder uniformity (Hicks 1991).

Hydrated lime is also determined to increase the asphalt mixture's stiffness by increasing the asphalt binder's viscosity. This property can help reduce the cohesion-related moisture damage in asphalt mixtures. The use of hydrated lime in asphalt mixtures was also determined to improve the fracture resistance of asphalt mixtures (Little, et al. 2006). A study conducted by Oregon State University (Kim et al. 1995) for the Oregon Department of Highways showed that adding lime into asphalt mixtures could significantly improve both fatigue cracking and rutting resistance of asphalt mixtures. The results also indicated that the performance improvement created by lime is significantly higher than the improvement created by liquid antistrips. However, additional testing and research with the asphalt mixtures used in Oregon today (including polymer-modified asphalt mixtures) are required to validate or falsify this conclusion since liquid antistrip technologies have also been improving within the last 27 years.

Lime is most commonly used in powdered form, but concerns about air-borne particles have encouraged the development of pelletized lime. If pellets are to be used, research has shown that mixing the pellets with the aggregate and water before drying provided the best results (Trejo et al. 2014). However, powdered and pelletized lime are used in large volumes and require substantial storage facilities and increased transportation costs. The large storage requirements, transportation costs, and health concerns of lime provide significant incentives to explore alternative additives for increasing the moisture resistance of asphalt concrete.

2.2.4 Liquid Anti-Strips (Chemicals)

Liquid antistripping additives can be mixed into HMA to strengthen the binder-aggregate adhesion and therefore reduce moisture susceptibility. Chemical additives work by changing the binder's chemical composition and electrical charge to promote uniform, complete coverage and, consequently, strong adhesion (Diab et al. 2017). However, results can vary widely depending on the binder type and aggregate mineralogy (Trejo et al. 2014; Anderson et al. 1982), and testing should be performed to optimize for locally available aggregate stockpiles. Research has shown that chemical antistripping additives exhibit a wide range of chemical stability over long exposure to high heat (Hicks 1991), reducing their efficacy. This reason further strengthens the importance of thorough testing prior to the implementation of chemical antistripping additives.

Before adding asphalt binder during mixture production, additives (surfactants) are mixed with the aggregates to reduce aggregate surface tension and move absorbed water to achieve better bonding between the asphalt binder and aggregate surface (Kennedy et al. 1983). However, mixing the additive with the binder is generally accepted to be a more economical, practical, and generally accepted method (Watson et al. 2013).

Research studies comparing the effectiveness of lime and liquid antistrips in reducing moisture susceptibility have variable conclusions. A study conducted by Sebaaly et al. (2007) showed that asphalt mixtures treated with lime (1.0% by dry weight of aggregate) had better moisture resistance than the mixtures treated with liquid antistrips (The Unichem-RAA04013: 0.5% by weight of binder). Another study from the same research group (Souliman et al. 2014) showed that liquid antistrips could improve the long-term cracking resistance of the asphalt mixtures by reducing the speed of oxidation-related binder aging. It was also concluded that although using lime in the asphalt mixture stiffens that asphalt binder and improves the rut resistance, it does not reduce the cracking resistance of the asphalt mixture.

A study conducted by Bennert and Venkiteela (2015) showed that some brands of the liquid antistrip agents were increasing the moisture susceptibility of the asphalt mixture while most of them increased the moisture resistance. In addition, some liquid antistrips might improve the rutting resistance, while some make it worse. These results showed that not all liquid antistrip technologies are equal, and some may provide worse results than others. For this reason, the type and brand of the liquid antistrips should also be reported when the conclusions regarding liquid antistrips are provided in research studies (although most scientific journals do not allow releasing the brand of a commercial product).

2.2.5 Improved Asphalt Mixture Production Process

Considering the time and resources invested in the design, testing, and optimization of asphalt mixtures and the scale of which they are implemented, it is important to accurately mirror design requirements when upscaling to the production phase. Asphalt binder content, aggregate gradation, additives, and production temperatures each play an important role in the long-term durability of asphalt concrete, each influencing the outcome of in-place density, impermeability, and overall moisture susceptibility. Although variance is inevitable during production, testing procedures exist to ensure mixture control. Testing is performed on a quantitative basis by extracting a sample per unit weight produced, or on a time basis, with samples extracted on a scheduled interval (Lundy 2001). Through quantitative sampling, a mixture can be tracked from production to lay down to identify issues within production or storage.

2.2.6 Asphalt Concrete In-Place Density and Reduced Permeability

Research has shown that the compacted density of asphalt concrete has a significant impact on moisture susceptibility. Asphalt density is commonly expressed in terms of air void content and can range from less than 4% to over 20%. Low air void content (less than 5%) reduces water infiltration, thereby increasing moisture resistance. High air void content (more than 15-20%) allows water to drain freely through the asphalt and results in minimal moisture-related failures (mostly in an open-graded permeable asphalt layer). Air void content between 5-15% allows water infiltration without the ability to drain freely, which can trap water in the asphalt causing accelerated stripping and reduced lifespan (Diab et al. 2017). Given the above considerations, it is important that in-place density is reduced to less than 5% air void content. Common practice is to compact 6-7% air voids during construction, then assume secondary compaction over time by traffic to the desired 4-5% air void content. However, this secondary compaction may not always reduce air void content to the required degree to ensure water impermeability.

Different methods and technologies have been used by ODOT to conduct verification tests to approve the quality of the construction methods and materials. Payment for materials and construction is made by following a percent within limits (PWL) specification. Pay factors for asphalt materials are determined by using the test results for asphalt content (26%), aggregate gradation (26%), and in-place density (48%). ODOT is currently in the process of changing the specifications to reduce the asphalt content tolerance from ± 0.5 % to ± 0.35 % to improve production precision and quality. There has also been an ongoing effort to develop methods to increase in-place density to improve the rutting and cracking performance of asphalt pavements (Coleri et al. 2017; Sreedhar and Coleri 2018). These efforts are expected to result in asphalt surfaced pavements with high-density levels and less permeability in Oregon. Increased density is also expected to reduce moisture-related failures.

The laydown phase of asphalt concrete construction is one of the most important, and quality can vary widely without proper care. Construction quality is generally measured in terms of density and smoothness during construction. Smoothness is an important consideration due to the public's perception of quality closely relating to this metric, while in-place density is important due to its correlation to moisture susceptibility and durability (Lundy 2001). As discussed in previous sections, compacting asphalt concrete to 6% air voids or less results in decreased permeability, increased moisture resistance, and a longer expected lifespan (Diab et al. 2017).

Many state highway agencies employ incentives/disincentives to help ensure quality control. A 1999 survey of eleven agencies reported nearly all agencies felt that a quality assurance program increased the quality of work (Lundy 2001). Oregon was part of the 1999 survey and continues to offer incentives to construction companies for meeting expectations.

2.2.7 Roadway Geometry and Drainage

Moisture damage from the introduction of high internal water pressures generated by traffic is a major detriment to pavement longevity, especially with standing water in the pavement layers. For this reason, roadway geometry, construction of proper superelevation on the roadway for effective water removal, and functioning drainage facilities can be considered to be one of the most, if not the most, important factors that are controlling moisture-related failures on roadways. Many studies have correlated poor subsurface drainage to accelerated asphalt pavement degradation (Hicks 1991; Scholz and Rajendran 2009; Brown 2010). To mitigate excessive moisture damage, adequate drainage of the pavement surface and surrounding area is necessary. Effective drainage is achieved by implementing multiple systems working in tandem to remove excess water and prevent pooling at or around the pavement surface.

There are many factors behind water drainage, but all begin with shedding water away from the pavement by sloping the surface longitudinally or transversely. Water is then directed into ditches, pipes, storm drains, or edge drains and transported away from the road (Scholz and Rajendran 2009). As discussed in the previous section, pavement density also plays a vital role in keeping the functionality of the drainage system by moving the water on the pavement surface without having any significant infiltration into the layers. It should be noted that studded tire damage might be another critical factor reducing the effectiveness of pavement drainage systems. Damage along the wheel paths can hold the rainwater and prevent it from flowing into the drainage facilities on the side of the roadway.

2.2.8 Moisture Related Delamination

The tack coat bond is known to dictate the longevity of asphalt pavements. Proper interlayer bonding prevents successive pavement layers from acting independently of one another and creating non-uniform stress and strain profiles in the pavement structure (FHWA 2016). Current structural pavement design methodologies assume that the pavement structure behaves monolithically (100% bonding between all pavement layers), which testifies to the importance of proper interlayer bonding using tack coat (FHWA 2016). Poor bonding between pavement layers can result in various pavement failures such as slippage cracking, debonding, and early fatigue cracking, all of which contribute to a reduced pavement fatigue life (Al-Qadi et al. 2012). King and May (2004) purport that a 50% reduction in pavement fatigue life can be expected when tack coat bond strength is reduced by 10%.

Coleri et al. (2017; 2020) indicated that tack coat quality, uniform tack coat application during construction, avoiding/minimizing tracking (the pick up of tack coat emulsions by construction vehicle tires), and proper cleaning of the milled surface before tack coat application are the major factors controlling the bonding between pavement layers. Based on the forensic investigations (trenching and coring) conducted on roadway sections with stripping issues, Scholz and Rajendran (2009) found that water infiltrating through the pavement layers can be trapped between the pavement lifts if the layers are not properly bonded. Standing water between the pavement lifts along the tack coat bonds will create high internal water pressures with the application of heavy truckloads, which separates the layers further and result in severe delamination and fatigue cracking. To combat this issue, it is important to make sure that tack coats are applied by following the important factors controlling tack coat bonding and performance [described in detail in Coleri et al. (2020)] to avoid any water getting in between the pavement lifts. Asphalt layer density, roadway geometry, and drainage are the factors that control the movement of water in between the pavement lifts, ultimately resulting in delamination and fatigue cracking failures.

2.3 TESTING AND CONDITIONING METHODS FOR MOISTURE SUSCEPTIBILITY QUANTIFICATION

A variety of conditioning methods have been developed over the years to mimic the long-term aging effects and conditions of asphalt pavements in the field, specifically due to moisture, heat, and traffic. The following methods were developed specifically to evaluate the moisture susceptibility of asphalt pavements, through either qualitative or quantitative analysis. While these conditioning methods have helped quantify the moisture susceptibility of asphalt pavements more accurately over the years, more research will be necessary to develop methods that will simulate the effects of actual exposure conditions of the field in an accelerated way. Some of the most common conditioning methods are discussed in this section.

Numerous testing methods have also been developed, modified, and implemented with the intent to quantify the effects of aging, moisture, or other environmental conditions on the strength and durability of asphalt concrete mixtures. These tests are generally conducted on specimens conditioned with different standard methods. Solaimanian et al. (2003) divided those tests and conditioning methods into two categories: moisture sensitivity tests conducted on loose and compacted asphalt mixtures. Table 2.1 and Table 2.2 provide the tests' names and specifications for loose and compacted asphalt materials, respectively. This section summarizes commonly used testing methods over the last four decades. The most promising tests with the conditioning methods are summarized in this section.

Table 2.1: Moisture-sensitivity Tests on Loose Samples (Solaimanian et al. 2003)

Test Method	ASTM	AASHTO	Other
Methylene Blue			Technical Bulletin 145, Ohio Department of Transporation (2002)
Film Stripping			California Test 302
Static Immersion	D1664* (1980)	T 182 (1984)	
Dynamic Immersion			No standard exists
Chemical Immersion			Standard Method TMH1 (Road Research Laboratory 1986, England)
Quick Bottle			Virginia Highway and Transportation Research Council (Maupin 1980)
Boiling	D3625 (2020a)		Tex 530-C (1999), Kennedy et al. 1984
Rolling Bottle			Isacsson and Jorgensen, Sweden, 1987
Net Adsorption			SHRP-A-341 (Curtis et al. 1993)
Surface Energy			Thelen 1958, HRB Bulletin 192, Cheng et al. AAPT 2002
Pneumatic Pull-Off			Youtcheff and Aurilio (1997)

Table 2.2: Moisture-sensitivity Tests on Compacted Samples (Solaimanian et al. 2003)

Test Method	ASTM	AASHTO	Other
Moisture Vapor Susceptbility			California Test 307, Developed in late 1940's
Immersion-Compression	D1075 (2011b)	T 165 (2002)	ASTM STP 252 (Goode 1959)
Marshal Immersion			Stuart 1986
Freeze/thaw Pedestal Test			Kennedy et al. 1983
Original Lottman Indirect Tension			NCHRP Report 246 (Lottman 1982); Transportation Research Record 515 (1974)
Modified Lottman Indirect Tension		T 283 (2014)	NCHRP Report 274 (Tunnicliff and Root 1984), Tex 531-C (1999)
Tunnicliff-Root	D4867 (2022b)		NCHRP Report 274 (Tunnicliff and Root 1984)
ECS with Resilient Modulus			SHRP-A-403 (Al-Swailmi and Terrel 1994)
Hamburg Wheel Tracking			Tex-242-F (2021)
Asphalt Pavement Analyzer			Pavement Technology Inc., Operating Manual
ECS/SPT			NCHRP 589 (Solaimanian, et al. 2007)
Multiple Freeze/thaw			No standard exists

2.3.1 Boiling Method

The boiling method (ASTM D3625, 2020a) immerses a loose sample of prepared asphalt mixture into boiling water for 1-10 min. After boiling, the sample is visually inspected for binder stripping in an attempt to estimate moisture susceptibility (Hicks 1991). Though the boiling method is inexpensive, easy to conduct, and useful for quick screening of additive efficacy, there are some shortcomings. While some researchers have found consistent correlations between boiling method test results and the field performance of antistripping agents, it also fails to identify aggregates' antistripping performance or correctly predict the antistripping performance of lime (Hicks 1991). Parker and Gharaybeh (1987) found that the boiling test cannot identify the moisture resistance benefits of using lime in the asphalt mixture. However, in another study, Parker and Wilson (1986) found that liquid antistrips and lime benefits could be successfully identified when the Texas Boiling Test was used. This conclusion was later supported by another study conducted by Lee and Al-Jarallah (1986). However, it was concluded in these research studies that visual analysis is subjective and therefore subject to variance depending on the person conducting the inspection. Other problems include the inability to test compacted asphalt, high dependency on binder viscosity, and a lack of strength analysis (Tsai et al. 2016).

Recent research suggested that digital image analysis could extract quantifiable stripping data from the boiling method. In a recent study, Tayebali et al. (2019) used a colorimeter CR400 device (a handheld device measuring and quantifying the color of materials) to quantify the change in color of loose asphalt mixture due to boiling. The device measures the color of the material by following the ASTM E284-22 (2022a) specification. An L* reading from the device was used to quantify the color of the boiled and unboiled asphalt mixtures. Loose asphalt mixtures were prepared with a PG64-22 asphalt binder. The Evotherm antistrip additive with a dosage of 0.5% by weight of the asphalt binder was used to prepare "antistrip" mixtures. Control mixtures were prepared by following the same method as the antistrip mixes without mixing any antistrip additives. Limestone and granite aggregates from different sources were used in the study. All mixes were dense-graded mixtures. The limestone mix had a nominal maximum aggregate size (NMAS) of 19.0mm, while the granite mix had an NMAS of 9.5mm. A stripping damage ratio was calculated based on the L* values, which are correlated with the specimen's color, measured before and after boiling. Based on the results, it was concluded that the stripping damage ratio calculated by the boiling method and the colorimeter measurements could quantify the benefits of using the Evotherm additive. Based on the results of several additional tests conducted in the study, the boiling test with a colorimeter was recommended as a cost-effective and practical method to quantify the impact of antistrip additives on moisture susceptibility.

2.3.2 Moisture Induced Stress Tester (M.I.S.T.)

The Moisture Induced Stress Tester (M.I.S.T.) is a relatively new device manufactured by the company InstroTek for testing HMA mixtures for moisture susceptibility. The M.I.S.T. aims to quantify HMA moisture susceptibility by simulating the cyclic loading conditions experienced in the field by vehicular loading. The test accelerates the damage rate by raising the water temperature to around 144ºF. The test is fully automated and takes approximately four hours (Cross et al. 2013).

Compacted samples are securely fastened inside the tank before filling the tank with water. Once the tank is filled with water, sealed, and the desired temperature is reached, a hydraulic cylinder begins pressurizing and depressurizing the tank repeatedly between 0 and 40 psi (while higher pressures are also possible) for the duration of the test. This cyclic pressurization is designed to simulate the cyclic nature of truck tires rolling over wet asphalt concrete by forcing water into the air voids under pressure, relieving that pressure, then repeating for up to 50,000 cycles. After the conditioning is complete, the samples are measured for post-test specific gravity and indirect tensile strength and compared with pre-conditioned samples (InstroTek 2011; Tsai et al. 2016).

Due to M.I.S.T. being relatively new, research on the accuracy of predictability of moisture susceptibility is limited. However, the test is promising in that some research has shown similar or better predictability of moisture susceptibility versus AASHTO T 283 (2014) while taking significantly less time. Dave et al. (2018) concluded that the use of M.I.S.T. conditioning with dynamic modulus testing provided results that are correlated with the expected field moisture resistance of asphalt mixtures.

Tayebali et al. (2019) concluded that the AASHTO T 283 (2014) and MIST conditioning methods are time-consuming, and the adhesion-related moisture failures cannot be simulated using these methods. On the other hand, it was concluded that the boiling test combined with the colorimeter measurements is an effective method to identify adhesion-related moisture failures. It was also concluded that MIST and the boiling tests could be used together to characterize both cohesion and adhesion-related failure mechanisms of asphalt mixtures.

2.3.3 Immersion-Compression

The static-immersion method (AASHTO T 182, 1984) entails submerging a loose HMA sample in water for 16-18 hours before observing the total visible area of the aggregate surface that retains the coating of asphalt binder. This test attempts to estimate stripping potential and, therefore, moisture susceptibility. However, this standard is obsolete as of 2007 and was replaced by the vacuum saturation test.

The immersion-compression test (ASTM D1075, 2011b) measured the reduction in asphalt compressive strength resulting from water damage (created by the immersion conditioning described in the previous paragraph). The test compared compressive strength results between unconditioned specimens against specimens that had been immersed in water for varying lengths of time. The test produced an index of retained strength, expressed as a percent of the unconditioned sample compressive strength. ASTM D1075 was withdrawn without replacement from ASTM standards in 2019 due to lack of industry use (ASTM 2011b) and is therefore not recommended.

2.3.4 Lottman Test

Asphalt concrete roads experience high levels of internal water pressure in wet conditions due to vehicular traffic, the action of which forces water into the pores of asphalt concrete. Vacuum saturation, as performed in the Modified Lottman Test (AASHTO T 283, 2014), attempts to replicate the deep moisture penetration experienced in the field by submerging compacted HMA samples into a pressurized water bath. AASHTO T 283 (2014) is used by ODOT and many other state agencies to evaluate moisture susceptibility, sometimes in conjunction with the Hamburg wheel tracking test (Scholz and Rajendran 2009). Though vacuum saturation is used in multiple test standards, finite element analysis has shown that moisture saturation may not be consistent throughout samples, therefore producing inconsistent moisture susceptibility estimates and a lack of repeatability. Researchers have also speculated that the vacuum's constant pressure applied to the samples is unrepresentative of field conditions generated by traffic and would be better simulated with a cyclic load (Cross et al. 2013).

The Lottman test (NCHRP 246, 1982) was developed under the National Corporative Research Program (NCHRP) in an effort to quantify the strength of specimens subjected to three different conditioning methods, (a) no conditioning, (b) vacuum saturation only, and (c) vacuum saturation with a freeze/thaw cycle (Cross et al. 2013; Lottman et al. 1974). Lottman proposed that moisture damage resulted from high pore pressures generated by traffic and thermal expansion. The test was developed to simulate high pore pressures in the asphalt concrete microstructure by saturating asphalt samples before freezing, then testing to determine the tensile strength of conditioned samples against unconditioned samples (Aschenbrener 1993). The air void content of the conditioned and unconditioned samples was specified to be anywhere between 3% to 5%, but research showed tensile strength ratios vary ±20% due to the variability of air voids between samples. Therefore, it was recommended that both conditioned and unconditioned samples have equal air void content (Aschenbrener 1993). This recommendation was eventually adopted in modified test standards of the original Lottman test.

Lottman finalized the test procedure in 1982 and was then used with varying degrees of modification in the years following. Some researchers performed the procedure with multiple freeze-thaw cycles, but this additional conditioning produced substantial variability in results and overestimated moisture susceptibility (Aschenbrener 1993). Ultimately, subsequent research led to the modification of the original Lottman test, and a new test standard was created, AASHTO T 283 (2014).

2.3.5 Modified Lottman Test

The original Lottman test was extensively used and modified before evolving into a new standard in 1985, known as the modified Lottman test (AASHTO T 283, 2014). This test combines elements from the Lottman test (NCHRP 246, 1982) and the Tunnicliff and Root test (ASTM D4867, 2022b). AASHTO T 283 (2014) specifies two groups of three samples with equal air voids between 6.5% to 7.5%. One group is unconditioned as a control group, while the second group is vacuum saturated to 70-80 percent saturation. The conditioned group is then subjected to a 16-hour minimum freeze-thaw cycle (optional). The conditioned samples are then subjected to a 140ºF hot water bath for 24 hours, after which the unconditioned samples are placed into waterproof bags, placed with the conditioned samples, and both subjected to a 77ºF water bath for 2 hours. Both groups are then tested for indirect tensile strength, and a tensile strength ratio (TSR) is obtained between the conditioned and control groups (AASHTO T 283, 2014). The Modified Lottman test adapted a 2in/min loading rate rather than the slower 0.065in/min loading rate used in the original Lottman method. This change was incorporated into the new specification to be able to run experiments with the Marshall stability testers that were available in most laboratories. A TSR of 0.80 or higher is recommended (Cross et al. 2013).

The modified Lottman test attempts to quantify the effect of water on the tensile strength of an asphalt mixture but is often used to compare the impact of anti-stripping agents on moisture susceptibility. The test can also be used to determine the optimum volume of anti-stripping agents necessary to maximize moisture resistance. Test procedures are nearly identical to the original Lottman test, with minor modifications to air void content range, saturation level, freeze length, and loading rate for the indirect tension test. Research of the original Lottman test had recommended equal air void content between conditioned and unconditioned samples, and this recommendation was implemented as a requirement in the modified Lottman test. However, research has shown that the modified Lottman test lacks repeatability and sometimes produces invalid results. Finite element analysis research has shown that moisture saturation may not be consistent throughout samples, therefore producing inconsistent moisture damage and lack of repeatability. Researchers have also speculated that the constant load applied to the samples by the vacuum tank is unrepresentative of field conditions generated by traffic and would be better simulated with a cyclic load (Cross et al. 2013; Solaimanian et al. 2007).

2.3.6 Tunnicliff and Root Method

The Tunnicliff and Root method (ASTM D4867, 2022b) is similar to the modified Lottman test (AASHTO T 283, 2014), but with some differences, such as an optional freeze-thaw cycle and slightly different conditioning times (curing of the loose mixture at 60°C in an oven for 16 hours was removed). ASTM D 4867 (2022b) specifies two groups of three samples with equal air voids between 6% to 8%. One group is unconditioned as a control group, while the second group is vacuum saturated to 70-80 percent saturation. The conditioned group is then subjected to an optional 16-hour freeze-thaw cycle. The conditioned samples are then subjected to a 140ºF hot water bath for 24 hours, then soaked in a 77ºF water bath for one more hour. The unconditioned samples are placed into waterproof bags before spending 20 minutes in a 77ºF water bath. Both groups are tested after their respective water baths for indirect tensile strength, and a TSR is obtained between the conditioned and control group. A TSR of 0.80 or higher is generally recommended (Cross et al. 2013).

Researchers using this test have produced varying thresholds of moisture susceptibility, making it challenging to use confidently without conducting comprehensive testing and research (Trejo et al. 2014). NCHRP Project 9-13 (Epps et al. 2000) investigated the effectiveness of AASHTO T 283 (2014) in identifying the impact of saturation levels, freeze-thaw cycles, and compaction levels on moisture susceptibility. It was concluded that AASHTO T 283 (2014) is not a reliable test for identifying the impact of those factors. It was also concluded that the TSR criterion should be modified to make the process work. Kanitpong and Bahia (2006) also concluded that TSR results are not correlated with the actual field moisture susceptibility of the mixes. However, it should be noted that field moisture resistance is not just controlled by the mixture properties, as also described in Section 2.2. Drainage, asphalt concrete density, construction variables, and production variability also control the moisture resistance. For this reason, studies directly comparing TSR values to field moisture resistance to evaluate the effectiveness of AASHTO T 283 (2014) can be misleading.

2.3.7 Environmental Conditioning System (ECS) and Dynamic Modulus and Resilient Modulus Testing

The Environmental Conditioning System (ECS) was developed at Oregon State University in the SHRP-A-403 research project (Terrel and Al-Swailmi 1994). Alam et al. (1998) later modified and improved the system for more straightforward implementation and more accurate results. A sample with a 102mm diameter and 102mm height was placed in the ECS chamber with a rubber membrane around it. The sample was then subjected to repeated loading, cyclic moisture conditioning (similar to M.I.S.T. – See Section 2.3.2), and different temperature levels. Resilient modulus or dynamic modulus tests were conducted with the samples with and without conditioning to quantify the resistance of the mix to the conditioning method. Solaimanian et al. (2003) indicated that ECS conditioning effectively simulates field conditions by pushing the water into the asphalt microstructure and simulating high pore pressures created by truck loads. However, it was also concluded that the correlation between the field moisture susceptibility and the test results from ECS conditioned samples is not higher than the correlation between field performance and the test results for the samples tested with AASHTO T 283 (2014). In addition, the cost of the system is significantly higher than the AASHTO T 283 (2014) setup. AASHTO T 283 (2014) also has a more practical conditioning process than the ECS.

The NCHRP Report 589 Solaimanian et al. (2007) compared the effectiveness of TSR, HWTD, and ECS/Dynamic Modulus tests in quantifying the moisture resistance of asphalt mixtures. It was concluded that ECS/Dynamic Modulus is the most effective method, while the TSR and HWTD methods have equal effectiveness. More details about this study are provided in the next section for HWTD.

2.3.8 Hamburg Wheel-Tracking Device

The Hamburg Wheel-Tracking Device (HWTD) was developed in the 1970s to evaluate the rutting resistance of HMA and has been standardized under AASHTO designation T 324 (2019b). HWTD simulates the field conditions of traffic with up to 20,000 passes of a 158-pound steel wheel over samples submerged in a 50ºC water bath. Rut depth is continuously measured using linear variable displacement transducers (LVDT) over the roughly seven-hour duration of the test. Because the samples are submerged in water during the test, the HWTD is often used to evaluate moisture susceptibility by identifying a stripping inflection point, indicated by a sharp increase of rut depth over wheel passes. Figure 2.3 shows an example of typical results obtained from the HWTD. The test provides information related to the total rut depth, post-compaction, creep slope, stripping inflection point, and stripping slope of the asphalt concrete sample (Yildirim et al. 2007; Tsai et al. 2016). Stripping inflection point and stripping slope are generally used to quantify the moisture resistance of asphalt mixtures. It was assumed that the repetition at which the sample starts to fail at a significantly faster rate is correlated with the field stripping resistance of the mix. However, this change in slope can also indicate tertiary flow rutting and not really be correlated with moisture susceptibility (Tsai et al. 2016). In addition, according to the HWTD test results, an asphalt mixture with high rutting resistance (a mix with a stiffer binder, lower binder content, lower VFA, higher dust content, etc.) may appear to be a mixture with high moisture resistance due to the slow deformation accumulation rate, although the mix can have poor field moisture resistance (Tsai et al. 2016). As of 2013, HWTD is used by many state highway agencies to determine HMA rutting resistance, with only a few using the test to evaluate stripping (Cross et al. 2013). However, several state highway agencies started to evaluate HWTD as a moisture susceptibility test for asphalt mixtures.

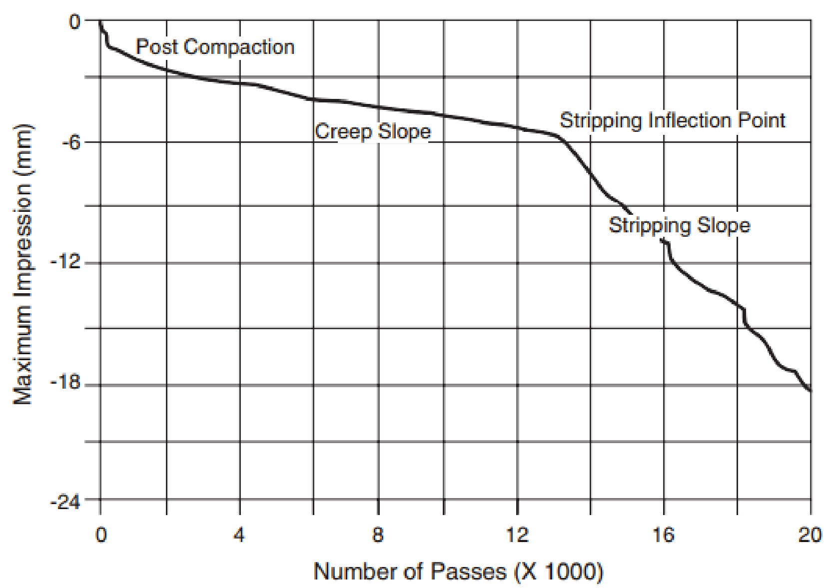


Figure 2.3: Typical HWTD test results (Yildirim et al. 2007)

Despite the widespread use of HWTD, research has identified some disadvantages. The current specification allows the use of two sample types, either a slab or cylindrical specimen. In a study from 2016, Tsai et al. recommended only a single sample type be used to be consistent and avoid specimen shape-related bias in the test results, but the AASHTO specification still allows both sample types. In the same study, it was found that the calculated average rut depth can vary depending on selected measurement stations along the wheel path, resulting in different rutting curves.

Finally, AASHTO T 324 (2019b) makes it difficult to identify whether the damage is caused by rutting, moisture, or both. Extensive testing with the HWTD conducted in Europe suggested that the test should only be used for rutting evaluation, as it has a minimal indication of moisture susceptibility (European Committee for Standardization (ECS), 2020 - ‘‘Small size device”, Procedure B conducted in the air; Tsai et al. 2016). Lu (2005) also concluded that the total measured surface deformation from the HWTD is a combination of both moisture and rutting-related permanent deformation. Lu (2005) recommended conducting both dry and wet HWTD tests to be able to separate the stripping-related deformation from the rutting-related deformation to quantify the moisture susceptibility of the asphalt mixture. However, it was noted that most of the HWTD tests conducted without water heating take a longer time to fail. In addition, running both dry and wet HWTD tests may reduce the practicality of the test method for moisture susceptibility quantification.

Izzo and Tahmoressi (1999) also concluded that the HWTD test is not able to quantify the moisture susceptibility of asphalt mixture when the mix is highly rutting susceptible and the test temperature is high (50ºC). A similar conclusion is also likely to be derived when the asphalt mixture is highly rutting resistant. It may be possible that a mixture with high moisture susceptibility with high rutting resistance might appear to be performing well in terms of stripping according to the HWTD test results when the stripping inflection point and stripping slope were used to evaluate the stripping resistance. It should be noted that the majority of the asphalt mixtures used by ODOT do not reach the tertiary deformation stage and do not provide a stripping inflection point or stripping slope (meaning high moisture resistance). However, the reason for not reaching the stripping inflection point is the high rutting resistance of ODOT mixes since some mixes used by ODOT exhibit poor moisture resistance in the field.

In NCHRP Report 589, Solamianian et al. (2007) investigated the effectiveness of TSR, HWTD, and ECS/Dynamic Modulus tests in quantifying the moisture resistance of asphalt mixtures. It was indicated that TSR (ASTM D4867, 2022b) and HWTD tests both failed to identify the field moisture susceptibility of 3 mixes out of 8 mixes, while the ECS/Dynamic Modulus test was able to identify 7 of the mixes correctly with just 1 false positive (See Table 2.3). Based on these results, it was concluded that ECS/Dynamic modulus test is the most effective experiment in identifying moisture susceptibility of asphalt mixture. However, it should be noted that the cost and practicality of the ECS/Dynamic modulus tests were not evaluated in this study.

The HWTD test results for the AR, WI, and MS mixes shown in Table 2.3 did not follow the reported field moisture resistance. The MS Chert mix had a significantly lower VFA than all other mixes (72.1), which is why the HWTD test predicted a good field performance for this mix while the actual field moisture resistance was low. The high rutting resistance of the mix dominated the results and did not provide a significantly high stripping slope, although the mix had poor moisture resistance in the field. AR Gravel mix also had a lower binder content than most other mixes (VFA for the AR mix was not provided in the report), which probably resulted in a "Good" moisture resistance from the HWTD results, although the mix performed poorly in the field in terms of moisture resistance. Similarly, HWTD test results for the WI Gravel mix (see Table 2.3) provided a "Poor" performance while the field moisture resistance of this mix was "Good". The potential reason for this false decision is the soft binder (PG58-28) used in the mixture. Asphalt mixture fails due to the soft binder and results in a high stripping slope, which was interpreted as "Poor" moisture resistance based on the HWTD testing and data processing methods for moisture susceptibility. All these results prove that HWTD test results are mostly controlled by the rutting resistance of the asphalt mixture, and the moisture susceptibility of the mix cannot be directly isolated from the final rutting curves. For this reason, HWTD may not provide reliable moisture susceptibility estimates in many cases.

Table 2.3: Summary of Results for all Mixes Tested Under Phase IA Research (Solaimanian et al. 2007)

Mix	TSR	HWTD	ECS/Dynamic modulus	Reported Field Performance
GA Granite	Poor	Poor	Poor	Poor
AR Gravel	Good[1]	Good	Poor	Poor
WI Gravel	Good	Poor	Poor	Good
MS Chert	Poor	Good	Poor	Poor
KY Limestone	Good	Poor	Poor	Poor
OK Sandstone	Poor	Good	Good	Good
PA Dolomite	Good	Good	Good	Good
WY Gravel	Poor	Poor	Poor	Poor

Note: [1]Underlined items refer to test predictions that did not match reported field performance.

2.4 SUMMARY

In this comprehensive literature review, the mechanisms behind asphalt moisture damage accumulation, moisture conditioning methods, and testing procedures were explored and evaluated by examining existing literature with the intent to identify improvements to ODOT's existing moisture susceptibility testing and conditioning methodologies.

The mechanisms controlling the loss of adhesion and cohesion in the asphalt mixture work in tandem to strip binder from aggregate and damage the binder phase in the asphalt concrete mixture, thereby accelerating pavement degradation (Hicks 1991), while the loss of adhesion was generally accepted to be the major factor controlling moisture susceptibility. Pavement distress from stripping severely reduces the designed lifespan of asphalt concrete and manifests as raveling, potholes, fatigue cracking, rutting, or bleeding.

Mixture level stripping has been shown to be influenced by aggregate mineralogy and chemistry, aggregate surface texture, asphalt binder chemistry, and the chemical and electrical interaction between the aggregate and binder (Bonaquist et al. 2007). Other factors that are expected to influence the moisture susceptibility of asphalt mixtures are: i) The use of additives; ii) Improved asphalt production process; iii) Higher in-place density and reduced permeability; iv)

Roadway geometry and drainage; and v) Moisture related delamination. The mechanisms of all these factors are discussed in this literature review.

Although the loss of adhesion and cohesion are the two major mechanisms causing moisture damage in asphalt mixtures, the delamination caused by water infiltrating through the pavement and damaging the bond between the two pavement layers is also a common mechanism for moisture-related pavement failure (Khosla et al. 1999; Scholz and Rajendran 2009*)*. For this reason, bonding between the pavement layers, density and permeability of the asphalt layers, roadway geometry, and the presence and functionality of drainage facilities along the roadway are also other factors controlling the moisture susceptibility of asphalt surfaced pavements. It is possible to have a roadway constructed with the least moisture susceptible asphalt mixture and still end up with severe stripping issues due to design and construction mistakes. For these reasons, it is not possible to eliminate moisture damage by just improving asphalt mixtures' moisture resistance. However, improving the mixture quality to reduce stripping is an important factor in combating moisture-related failures on roadways. In this study, only the asphalt mixture level moisture susceptibility was evaluated to determine the most effective and practical conditioning and testing method for Oregon.

A variety of conditioning methods have been developed over the years to mimic the long-term aging effects and conditions of asphalt pavements in the field, specifically due to moisture, heat, and traffic. The methods discussed in this literature review were developed specifically to evaluate the moisture susceptibility of asphalt pavements through either qualitative or quantitative analysis. While these conditioning methods have helped quantify the moisture susceptibility of asphalt pavements more accurately over the years, more research will be necessary to develop methods that will simulate the effects of actual exposure conditions of the field in an accelerated way. Some of the most common conditioning methods are discussed in this literature review.

Asphalt concrete roads experience high levels of internal pore pressure in wet conditions due to vehicular traffic, the action of which forces water into the pores of asphalt concrete. Vacuum saturation, as performed in the Modified Lottman Test (AASHTO T 283, 2014), attempts to replicate the deep moisture penetration experienced in the field by submerging compacted HMA samples into a pressurized water bath. AASHTO T 283 (2014) is used by ODOT and many other state agencies to evaluate moisture susceptibility, sometimes in conjunction with the Hamburg wheel tracking test (Scholz and Rajendran 2009). Though vacuum saturation is used in multiple test standards, finite element analysis has shown that moisture saturation may not be consistent throughout samples, therefore producing inconsistent moisture susceptibility estimates and a lack of repeatability. Researchers have also speculated that the vacuum's constant pressure applied to the samples is unrepresentative of field conditions generated by traffic and would be better simulated with a cyclic load (Cross et al. 2013).

Although the Boiling method (ASTM D3625, 2020a) is a subjective test for evaluating moisture susceptibility of asphalt mixtures, the use of recently developed imaging systems and methods can eliminate the subjectivity and quantify the adhesion-induced moisture susceptibility of asphalt mixtures (Tayebali et al. 2019). Test methods with MIST conditioning provided variable results. Some of the research studies in the literature suggested MIST to be used as a conditioning method for the AASHTO T 283 (2014) specification (Dave et al. 2018). On the other hand, some research studies pointed out the cost and practicality issues associated with the MIST conditioning methods (Tayebali et al. 2019).

Solaimanian et al. (2003) indicated that ECS conditioning effectively simulates field conditions by pushing the water into the asphalt microstructure and simulating high pore pressures created by truck loads. However, it was also concluded that the correlation between the field moisture susceptibility and the test results from ECS conditioned samples is not higher than the correlation between field performance and the test results for the samples tested with AASHTO T 283 (2014). In addition, the cost of the system is significantly higher than the AASHTO T 283 (2014) setup. AASHTO T 283 (2014) also has a more practical conditioning process than the ECS.

The Hamburg Wheel-Tracking Device (HWTD) was developed in the 1970s to evaluate the rutting resistance of HMA and has been standardized under AASHTO designation T 324 (2019b). Because the samples are submerged in water during the test, the HWTD is often used to evaluate moisture susceptibility by identifying a stripping inflection point, indicated by a sharp increase of rut depth over wheel passes. Stripping inflection point and stripping slope are generally used to quantify the moisture resistance of asphalt mixtures (See Figure 2.3). As of 2013, HWTD is used by many state highway agencies to determine HMA rutting resistance, with only a few using the test to evaluate stripping (Cross et al. 2013). However, several state highway agencies started to evaluate HWTD as a moisture susceptibility test for asphalt mixtures.

AASHTO T 324 (2019b) makes it difficult to identify whether the damage is caused by rutting, moisture, or both. Extensive testing with the HWTD conducted in Europe and the U.S. suggested that the test should only be used for rutting evaluation, as it has a minimal indication of moisture susceptibility (European Committee for Standardization (ECS), 2020 - ‘‘Small size device”, Procedure B conducted in the air; Tsai et al. 2016). Lu (2005) also concluded that the total measured surface deformation from the HWTD is a combination of both moisture and rutting-related permanent deformation. Lu (2005) recommended conducting both dry and wet HWTD tests to be able to separate the stripping-related deformation from the rutting-related deformation to quantify the moisture susceptibility of the asphalt mixture. According to the literature review conducted in this research project, HWTD may not provide reliable moisture susceptibility estimates in many cases since test results are controlled mainly by the rutting resistance of the asphalt mixture (See section 2.3.8).

Several research studies were conducted in the literature to determine the effectiveness of different testing and conditioning methods for moisture susceptibility evaluation. The majority of these research studies focused on correlating test results with the reported field moisture resistance of asphalt mixtures to determine the method's effectiveness. It should be noted that some of the conclusions from those studies can be misleading since the in-situ moisture susceptibility of asphalt mixtures is not just controlled by mixture properties. Drainage, asphalt concrete density, construction variables, and production variability also control the moisture resistance. For this reason, studies directly comparing TSR values to field moisture resistance to evaluate the effectiveness of AASHTO T 283 (2014) can be misleading.

3.0 ASPHALT MIXTURE PLANT SAMPLING AND GENERAL MIXTURE PROPERTIES

This section details the sampling of materials used in this study and the material properties of the sampled asphalt mixtures.

A total of ten asphalt mixtures with different designs were selected for the study on the recommendation from ODOT. The materials were collected from different regions of Oregon, representing designs for different environmental and traffic conditions. The mixtures were selected on the basis of availability, importance, and construction schedule. All mixtures evaluated in this study were produced in asphalt production plants and sampled right before construction. Thus, these asphalt mixtures were evaluated and compared based on only expected performance as there was no record of historical performance in terms of cracking and moisture susceptibility. The general properties of the production asphalt mixtures used in this study are shown in Table 3.1.

Table 3.1: Production Mixture Information for the Asphalt Mixtures Sampled from the Plant

ID	Highway ID	Mix Design Level	Binder Grade	RAP[1] (%)	AC[2] (%)	P_{be}[3] (%)	D/B[4] Ratio	Additive	Add.[5] %	VMA[6]	VFA[7]	Plant TSR[8]	Lab TSR		
													M1[9]	M2[10]	V[11]
M1	-	Level 4	PG 70-22ER	30	5.6	-	-	CONV[12]	-	-	-	-	96.05	92.98	92.65
M2	US 20	Level 3	PG 64-22	30	5.7	4.79	-	CONV	-	15.1	71	-	87.80	85.80	82.57
M3	I-5	Level 4	PG 70-22ER	30	5.5	4.99	1.6	EVO[13]	0.50%[14]	15.4	74	90	95.09	87.51	88.73
M4	US 97	Level 3	PG 64-28	30	6.3	4.25	1.7	LIME	1.0%[15]	15.8	75	84	96.63	95.22	87.32
M5	US 26	Level 3	PG 64-22	30	5.8	4.67	1.3	ArrMaz Ad-Here (L-AS[16])	0.25%[14]	14.7	73		-	97.71	86.89
M6	US 26	Level 3	PG 64-22	30	5.8	4.67	1.3	CONV	-	14.7	73	85.8	-	90.94	90.26
M7	Various	Level 2	PG 64-28	25	5.7	4.97	1.4	CONV	-	15.9	75	97	-	97.52	97.59
M8	US 97	Level 3	PG 64-28	28	5.6	4.66	1.4	EVO	0.36%[14]	15.6	75	92	-	92.70	92.36
M9	OR Lake 4-12 (1)	Level 3	PG 64-22	20	6.0	4.84	1.1	EVO	0.50%[14]	15.5	74	99.6	-	97.98	98.93
M10	I-5	Level 4	PG 70-22 ER	30	5.5	4.30	1.6	LIME	1.0%[15]	14.6	73	92	-	89.54	95.17

[1] Reclaimed asphalt pavement added by weight
[2] Asphalt content added by weight
[3] Effective asphalt content present by weight in the total mix
[4] Dust to binder ratio in the mix
[5] Additive
[6] Voids in mineral aggregate
[7] Voids filled with asphalt
[8] Tensile strength ratio

[9] MIST#1
[10] MIST#2
[11] Vacuum
[12] Conventional mix
[13] Evotherm
[14] Added by weight of the asphalt binder
[15] Added by weight of the mix
[15] Added by weight of the mix
[16] Liquid anti-strip

In Oregon, TSR is currently used as a pass/ fail test of moisture susceptibility (ODOT 2018). As can be seen from the table, all the mixtures have passed the TSR test (TSR > 80%). In the absence of historical performance records, the TSR values of the mixtures can be used as indicators of expected good (high TSR values) or bad (lower TSR values) performance against moisture susceptibility. However, ODOT has encountered moisture damage even in the mixtures with high TSR values in the past. There were also reported cases in which the asphalt mixtures with low TSR values performed better than those with higher TSR. Thus, the plant TSR values have not been

effective indicators of moisture susceptibility in some cases. It should also be noted that one of the objectives of this study is to check whether TSR correlates well with other indicators of moisture susceptibility, which can be some fundamental mixture properties or the results and parameters from other tests and methods.

As discussed earlier, the production mixtures were sampled from different locations in Oregon, which is evidenced by different mix design levels and binder grades used. While mix design level is an indicator of expected truck traffic and load level, the PG grade of the binder may be determined by the climate conditions (in terms of expected high and low-temperature levels). It can be seen from Table 3.1 that six of the mixes are Level 3 mixtures, three are Level 4 mixtures, and one is a Level 2 mixture. There is also a wide variation in the binder grades used in these mixtures. Four out of ten mixtures use PG 64-22, and three contain PG 64-28 binder grades. Both PG 64-22 and PG 64-28 are softer binder grades with the only difference in low-temperature cracking resistance. The -28°C grades are used for colder regions in Oregon. The remaining three mixtures are Level 4 mixtures and contain stiffer binder grades (PG 70-22) to account for heavy traffic. Moreover, all Level 4 mixtures also use polymer-modified binder (denoted by suffix ER).

Similar to binder grades, the sampled production mixtures also have a wide range of design and volumetric properties. Although the volumetric properties of each mixture lie within the allowable range specified by ODOT, differences in values are expected to have an impact on the mixture's performance against distresses, including moisture damage. Factors like asphalt content and aggregate properties play a significant role in controlling moisture damage, as discussed in Sections 2.2.1 and 2.2.2. One factor which is of particular importance is the RAP content present in the asphalt mixture. The source and gradation of RAP as well as the blending of RAP binder with the virgin binder, affect the performance of asphalt mixtures. Several studies have indicated that higher RAP contents result in lower rutting, higher cracking, and higher moisture resistance (Coleri 2017; Rahman and Hossain 2014). Table 3.1 shows that all except three mixtures have 30% RAP content. Mix 7, Mix 8 and Mix 9 have 25%, 28%, and 20% RAP, respectively.

In addition to volumetric properties and binder grades, the asphalt mixtures used in this study differ in one more aspect. Six of these mixtures have the presence of an additive. Mix 4 and Mix 10 have 1% lime (by weight of the mix), while Mix 3, Mix 8, and Mix 9 have, respectively, 0.50%, 0.36%, and 0.50% Evotherm in them (by weight of the asphalt binder). Mix 5 has 0.25% of an anti-strip liquid additive (by weight of the asphalt binder). The potential effect of these additives on asphalt mixtures' performance and moisture resistance is described in Sections 2.2.3 and 2.2.4. It can be noted here that the chemical warm mix additive Evotherm was used as a liquid anti-strip and compaction aid in the respective mixtures without any significant reduction in production and compaction temperatures. Thus, all ten mixtures were produced as HMA with mixing and compaction temperature within the conventional range irrespective of the type of additives used. It should be noted that according to Kumar et al. (2021), preparing WMA mixtures at high HMA mixing temperatures results in reduced cracking performance due to the excessive aging of the WMA mixture and the volatilization of some portion of the WMA additive. High mixing temperatures were determined to be nullifying the positive effects of warm mix additive on the cracking resistance of the mix. For this reason, SPR 826 (Kumar et al. 2021) recommended the use of manufacturer-recommended (lower) mixing temperatures for the production of WMA mixtures in Oregon.

4.0 LABORATORY INVESTIGATION

4.1 INTRODUCTION

The following sections discuss the moisture susceptibility evaluation methodology and the experimental plan for this study. This includes a detailed description of the conditioning methods and the tests selected in this research project.

4.2 RESEARCH METHODOLOGY AND EXPERIMENTAL PLAN

The main purpose of this study was to evaluate the selected asphalt mixtures for moisture susceptibility in the laboratory by following different testing and conditioning methods. Any moisture susceptibility evaluation method necessitates two steps. The first step is moisture conditioning, and the second step is some form of mechanical testing and/or analysis method. The first step (conditioning) is important for separating the unconditioned material from the conditioned material by a deliberate moisture intrusion. The second step measures the impact of the first step on the material. Since there exist various moisture conditioning and test methods, the testing plan for this study was divided into four phases. The first phase of the study aimed to select the most promising testing and conditioning protocol among the shortlisted methods while also evaluating the sensitivity of the test and conditioning methods to various factors. The second phase focused on examining the potential of the selected conditioning and testing protocols in separating "good" mixtures from "bad" mixtures for a larger variety of asphalt mixtures. In the third phase of the study, both compacted and loose mixtures were used to quantify the stripping in the mixtures by following a simpler test and measurement method. The fourth phase consisted of evaluating innovative and state-of-the-art test methods and technologies that had the potential to quantify and monitor the moisture susceptibility of asphalt mixtures. Each phase and the respective experimental plan are discussed in the subsequent subsections.

4.2.1 Phase I

This phase of the study was designed to compare different testing and conditioning methods in determining the moisture susceptibility of the asphalt mixtures. Four different mixtures were used for evaluation purposes. HWTT and IDT tests were selected to measure the impact of moisture conditioning on the test specimens prepared from the four mixtures. The IDT tests were used to obtain strength, CT-Index, and TSR values, and the HWTT tests provided rut depth information as well as other parameters extracted from the rutting evolution curve. Two types of conditioning methods were chosen in this study – vacuum conditioning (current conditioning method in AASHTO T 283, 2014) and MIST conditioning. All testing and conditioning methods are discussed in Section 2.3, and the procedures are further described in more detail in Sections 4.4 and 4.5. The MIST conditioning was carried out at 50ºC temperature and at two different pressures, 40 psi and 70 psi, to determine the impact of applied internal water pressure on the test results. In this report, the 50ºC-40 psi and 50ºC-70 psi MIST conditioning are referred to as MIST#1 and MIST#2, respectively. The HWTD tests were carried out at 50°C, 55°C, and 60°C in submerged conditions to determine the impact of test temperature on response. The experimental plan for Phase I of the study is shown below in Table 4.1.

Table 4.1: Experimental Plan to Compare Different Testing and Conditioning Methods for Moisture Susceptibility Evaluation

Mix No	Conditioning methods	Test methods	Temp.	Replicates	Total samples
M1 **M2**	Control – No conditioning Vacuum saturation (AASHTO T 283) MIST#1- 50ºC-40 psi MIST#2- 50ºC-70 psi	HWTT	50 ºC 55 ºC 60 ºC	4	192
M3 **M4**		IDT	25 ºC	3	48

4.2.2 Phase II

In Phase II of the study, the most effective combinations of the conditioning and test methods, identified based on the Phase I test results, were further experimented on the remaining six out of ten production mixtures. The purpose of this phase was to determine the ability of the selected testing and conditioning protocol to separate the expected "good" and "bad" performers. This phase of testing mixtures is also intended to validate the best conditioning and testing protocol for moisture susceptibility evaluation. A detailed experimental plan is shown in Table 4.2. Since MIST#1 conditioning with 40psi pressure was determined to result in moisture damage that is significantly less than vacuum saturation (AASHTO T 283, 2014) in Phase I, it was excluded from Phase II. MIST#2 conditioning with higher applied pressure was kept in the experimental plan since it provided moisture damage levels that are comparable to vacuum saturation. In addition to the test methods used in Phase I, the Phase II experimental plan also included the Resilient Modulus (RM) test, which is a non-destructive test conducted to determine the elastic modulus of the asphalt concrete materials.

Table 4.2: Experimental Plan to Identify the Best Conditioning and Testing Protocol for Moisture Susceptibility Evaluation

Mix No	Conditioning methods	Test methods	Temp.	Replicates	Total samples
M5 **M6** **M7**	Control – No conditioning Vacuum saturation (AASHTO T 283, 2014) MIST#2- 50ºC-70 psi	HWTT	50 ºC	4	72
M8 **M9**		IDT	25 ºC	3	54
M10		RM	25 ºC	3	54

4.2.3 Phase III

In a departure from the use of only compacted specimens and mechanical test methods, this phase of the study utilizes both compacted and loose production mixtures and a visual observation method. All mixtures except Mix 1 (due to unavailability) were included in the experimental plan of this phase, as shown in Table 4.3 below. A colorimeter device (a handheld camera system- CM-600D to quantify the color of objects. See Section 2.3.1) was used to measure the color of the boiled and unboiled asphalt mixtures to quantify the percentage stripping. Loose mixtures required for the test were collected from two separate buckets of each production mix to account for the variation in sampling, and two replicate samples were prepared from each bucket. The test method is described in Section 2.3.1, and the procedure is explained further in Section 4.5.4.

Table 4.3: Experimental Plan for Loose Mix Boiling Test on Loose Mixtures

Mix No.	No. of buckets	Replicates from each bucket	Total samples
M2 – M10 (total 9 mixes)	2	2	36

Other than the loose mix samples, colorimeter measurements were also taken from the surfaces of the broken halves of the IDT samples used in Phase II experiments. It is expected to have a lighter color on the moisture-conditioned samples due to stripping. The two halves of the fractured IDT sample were used. The experimental plan is shown in Table 4.4 below.

Table 4.4: Experimental Plan for Colorimeter Measurements on Fractured IDT Samples

Mix No.	No. of IDT cores		Total samples
	Control	Vacuum Conditioned	
M5 – M10 (total 6 mixes)	2	2	24

4.2.4 Phase IV

The testing in Phase IV was conducted based on the findings from the literature review and results of the experiments conducted in the previous three phases. The experiments conducted in this phase were not in the experimental plan discussed with the TAC. This phase was also not included in the original work plan. In this phase, three innovative test methods were explored for their potential as moisture susceptibility assessment methods for asphalt mixtures. The purpose of conducting these tests was to determine the feasibility of these experiments and technologies for moisture susceptibility evaluation. For this reason, the research efforts in this phase can be considered to be preliminary and should be investigated in more detail in a future research study.

The three evaluated test methods are: 1) HWTT under dry and wet conditions, 2) Moisture infiltration sensor technology, and 3) Electrical resistivity method. The goal was to examine the feasibility of these tests and technologies for moisture susceptibility evaluation by running preliminary experiments to determine their potential and without running comprehensive full-factorial testing at this stage. One core for each of the four mixes (Mix 2, Mix 8, Mix 9, and Mix 10) was used for the electrical resistivity testing. Four blocks with varying air voids prepared from Mix 2 were utilized for the moisture infiltration tests. The experimental plan for electrical resistivity and moisture infiltration tests is shown in Table 4.5.

Table 4.5: Experimental Plan to Determine the Feasibility of Electrical Resistivity and Moisture Infiltration Tests for Moisture Susceptibility Evaluation

Test Type	Mix No.	Target Air void	Total samples
Electrical resistivity	M2, M8, M9, M10	7 %	4
Moisture infiltration	M2	5%, 7%, 9%, 15%	4

HWTT tests were also conducted in wet and dry conditions to isolate the rutting-related deformation from the moisture and rutting-related portion (wet minus dry results). The area under the repetition versus rut depth curves for dry tests was subtracted from the area under the wet vacuum conditioned specimen tests to determine the moisture-related portion of the accumulated deformation. For the HWTT test under dry and wet conditions, two replicates of Mix 2, Mix 3, Mix 4, Mix 5, Mix 6, and Mix 9 were tested in dry and submerged (wet) conditions. For the wet tests, only vacuumed conditioned samples were used based on the findings presented in Section 4.6.1.2.1. The experimental plan for the HWTT tests is shown below in Table 4.6.

Table 4.6: Experimental Plan for HWTT Tests Under Dry and Wet Conditions

Mix No	Conditioning methods	Test methods	Temp.	Replicates	Total samples
M2	Dry – No prior conditioning Wet – Vacuum conditioned	HWTT	50 ºC	2	24
M3					
M4					
M5					
M6					
M9					

4.3 ASPHALT MIXTURE PREPARATION

Loose production mixtures sampled from different plants across Oregon were used for Plant Mixed and Laboratory Compacted (PMLC) specimen production in the laboratory. Loose production mixes collected from the plant before construction were stored in airtight buckets. In the laboratory, these buckets were then placed in the oven at 110°C for two hours. Uniform sampling of the mix was carried out using a mechanical splitter, as shown in Figure 4.1. The theoretical maximum specific gravity (G_{mm}) of the asphalt mixtures were determined. The required amount of asphalt mixture for samples of each mix type was determined (using the measured G_{mm} values), weighed to achieve 7% air-void content for different samples, and placed in the oven at the compaction temperature for two more hours. Cylindrical samples were compacted using the SGC in accordance with the AASHTO T 312 (2019c) specification.

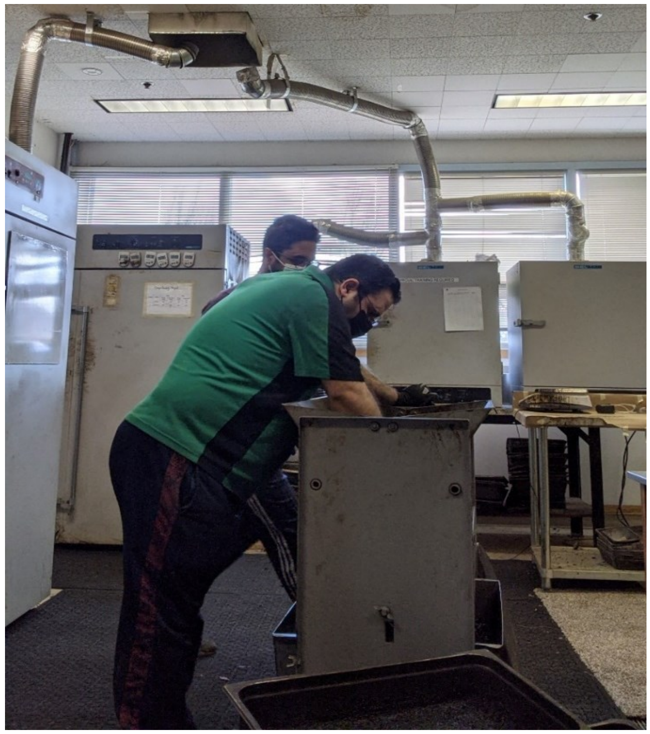


Figure 4.1: Mechanical splitting of asphalt mixtures

4.4 MOISTURE CONDITIONING METHODS

More detailed information and the literature on the moisture conditioning methods are provided in Section 2.3. This section summarizes only the conditioning processes directly followed in this research study.

4.4.1 Vacuum Saturation

In this method, specimen conditioning was performed according to AASHTO T 283 (2014) by immersing the specimens in water and exposing them to a vacuum for different treatment periods to achieve saturation levels ranging from 70% to 80%. The following procedure was followed to condition the specimens in this study:

- Place the specimen on the perforated spacer in the vacuum container.
- Fill the vacuum container with potable water at room temperature so that the specimen has at least 1 in (25mm) of water above its surface.
- Allow the vacuum to reach a gauge pressure of more than 25 in Hg (check on the vacuum gauge), then stop the vacuum pump. Slowly open the pressure release valve on the lid to reach the partial pressure of 20 in Hg and then close it.
- Hold this pressure for 5 min.

- Slowly remove the vacuum by opening the pressure release valve.
- Leave the specimen submerged in water for another 5 min.
- Open the lid and determine the mass of the saturated surface-dry specimen after the partial vacuum saturation.

If the specimens could not achieve saturation levels between 70% and 80%, they were conditioned again in the vacuum chamber by changing the duration of the pressure until the desired saturation level was reached. But if the specimens achieved saturation level above 80 percent, they were discarded. The vacuum-conditioned samples were placed in the water bath at 60°C for 24 hours before being tested in HWTT, and additionally in a water bath at 25ºC for two hours before the IDT test. Figure 4.2 shows the setup for vacuum conditioning used in this study. Equation 4-1 below was used to calculate the percentage of saturation.

$$\boldsymbol{S = \frac{J}{Va} * 100} \tag{4-1}$$

where:

S = Percent saturation

J = Mass of absorbed water

V_a = Specimen air voids

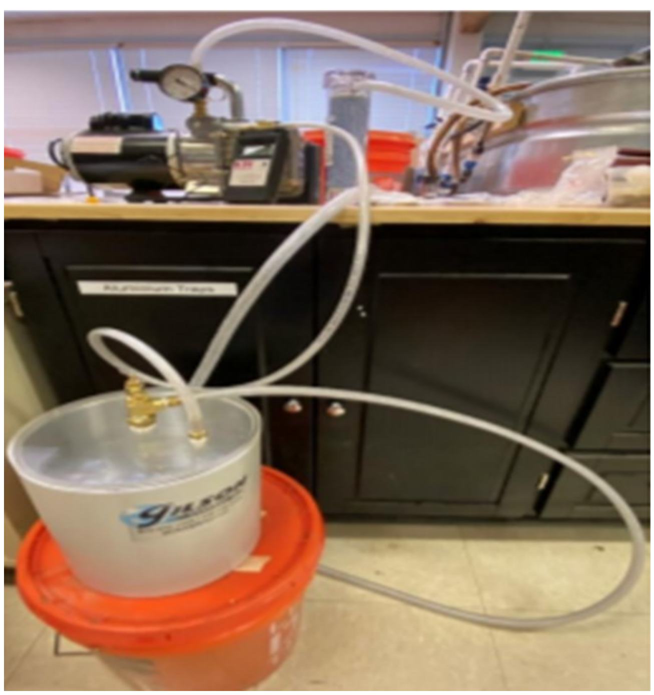


Figure 4.2: Setup used for vacuum conditioning

4.4.2 Moisture Induced Stress Tester (M.I.S.T.)

MIST was developed to simulate the conditions of repeated generation of pore pressure in a saturated pavement under traffic load (See Section 2.3.2). In this study, MIST moisture conditioning is performed using the MIST equipment shown in Figure 4.3.

Figure 4.3: Moisture-induced sensitivity tester (MIST) equipment (InstroTek 2011)

The procedure described in AASHTO T 283 (2014) was followed to test the MIST conditioned samples in this study. In Phase 1, the specimens were conditioned in MIST for 20 hours at 50°C followed by 3.5 hours of cyclic water pressure application (for 3,500 cycles). Two different pressures were used for this purpose – 40 psi and 70 psi. After conditioning in MIST, the sample was tested in HWTT for rutting and moisture susceptibility. However, for the IDT test for cracking, the conditioned samples were further conditioned in a water bath (25°C) for an additional two hours before testing.

4.5 TESTING METHODS FOR MOISTURE SUSCEPTIBILITY

More detailed information and the literature on the testing methods are provided in Section 2.3. This section summarizes only the testing processes directly followed in this research study.

4.5.1 Indirect Tension (IDT) Test & Modified Lottman Test – AASHTO T 283 (2014)

4.5.1.1 Modified Lottman Test – AASHTO T 283 (2014)

The modified Lottman or AASHTO T 283 (2014) test (see Section 2.3.5) includes the IDT test of moisture-conditioned samples to obtain the Tensile Strength Ratio (TSR). TSR is basically the ratio of the average tensile strength of unconditioned specimens ($S1$) to the average tensile strength of conditioned specimens ($S2$). Mixes with high moisture susceptibility are expected to have lower TSR values, while the mixes with lower moisture susceptibility should have higher TSR values. The tensile strength ratio (TSR) was calculated using Equation 4-2 as follows.

$$\boldsymbol{TSR = \frac{S2}{S1}}$$

(4-2)

where:

TSR = tensile strength ratio.

S1 = average tensile strength of unconditioned samples.

S2 = average tensile strength of conditioned samples

4.5.1.2 CT-Index

IDEAL Cracking Test (IDEAL-CT) was developed by Zhou et al. (2017) and is an extension of the indirect tensile strength test. Cylindrical specimens of 150 mm diameter and 50 mm height are subjected to a loading rate of 50 mm/min, and the test is carried out at 25 °C. The different parameters utilized in calculating the CT-index from the load-displacement curve of IDT are shown in Figure 4.4.

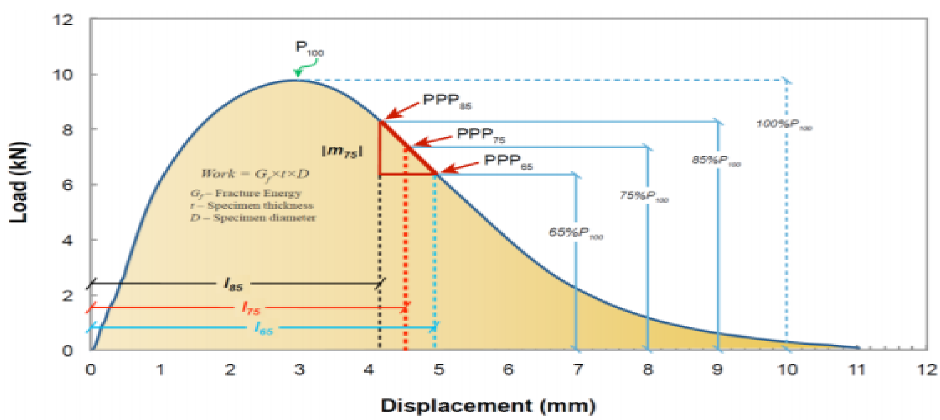


Figure 4.4: Typical load-displacement curves for IDEAL-CT (ASTM D8225, 2019a), Zhou et al. (2017)

The parameters shown in Figure 4.4 are explained below:

G_f: is the area of the load vs. vertical displacement curve divided by the area of the cracking face.

P/l: is the slope of the load-displacement curve.

D: diameter.

L_{75}: the displacement at 75% of the peak load after the peak.

m_{75}: the slope of the tangential zone around the 75% peak load point after the peak; and

t: the thickness of the specimen

CT-index is calculated using Equations 4-3 and 4-4 below:

For 62 mm thick specimens:

$$\boldsymbol{CTindex} = \frac{\mathbf{G_f}}{|\mathbf{m_{75}}|} \times (\frac{\mathbf{L_{75}}}{\boldsymbol{D}}) \tag{4-3}$$

For non-62 mm thick specimens:

$$\boldsymbol{CTindex} = \frac{\mathbf{t}}{|\mathbf{62}|} \times \frac{\mathbf{G_f}}{|\mathbf{m_{75}}|} \times (\frac{\mathbf{L_{75}}}{\boldsymbol{D}}) \tag{4-4}$$

For example, using M10 control R1 data, CT-index can be calculated as follows (required parameters are calculated using a code developed at OSU-AMaP):

For 50 mm thick specimens:

$$\boldsymbol{CTindex} = \frac{\mathbf{50}}{\mathbf{62}} \times \left(\frac{\mathbf{14.82}}{\mathbf{5.38}}\right) \times \left(\frac{\mathbf{6.39}}{\mathbf{150}}\right) \times \mathbf{1000}$$
$$\boldsymbol{CTindex} = \mathbf{94.6} \tag{4-5}$$

4.5.2 Resilient Modulus Test

The resilient modulus is defined as the ratio of stress to strain for an instantaneous load. It gives a measure of stiffness for the asphalt mixtures. In this study, the ASTM D7369 (2020b) specification is followed to conduct resilient modulus experiments. The test setup is shown in Figure 4.5.

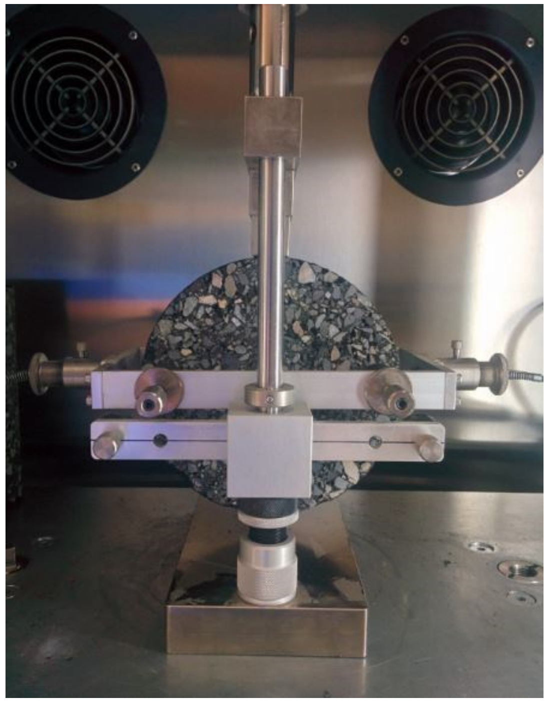


Figure 4.5: Resilient modulus test setup

Sample preparation and testing procedure:

- Cylindrical specimens of 150 mm diameter are produced using the SGC (AASHTO T 312, 2019c).
- The SGC core is cut into slices of 50 mm height using a high-precision saw. A minimum of three replicate samples are prepared.
- Samples are placed in the environmental chamber set at 25 ± 1ºC for 2 hours to ensure the specimen is at test temperature prior to beginning the test.
- At the end of the conditioning period, an initial vertical contact load is applied to the specimen. The contact load is 4% of the maximum load ($0.04P_{max}$) and is not less than 22.2 N but not more than 89.0 N.
- After applying the contact load, a cyclic load is applied to the specimen. The cyclic load is calculated using Equation 4-6 as:

$$P_{cyclic} = P_{max} - P_{contact}$$

(4-6)

- Resilient modulus is calculated automatically by the test software. It is essentially the ratio of peak stress to peak strain for every load cycle.

4.5.3 Hamburg Wheel Tracking Device (HWTD) Test

The Hamburg Wheel-Tracking Test (HWTT) system was developed to measure the rutting and moisture damage (stripping) susceptibility of an asphalt concrete sample. The HWTT follows the AASHTO T 324 (2019b) standard. According to the specification, either a slab or a cylindrical specimen can be tested. In this part of the study, tests were conducted by immersing the asphalt concrete sample in a hot water bath at temperatures 50°C, 55°C, and 60°C and rolling a steel wheel across the surface of the sample to simulate vehicular loading for Phase 1. All testing for Phase 2 was conducted at 50°C. Approximately 20,000-wheel passes are commonly used to evaluate the rutting and stripping resistance of a sample. However, in this study, the number of passes was increased to 30,000 to obtain a rutting evolution curve with three distinct stages (see Figure 2.3). The test details are described in Section 2.3.8.

As discussed previously in Section 2.3.8, the results obtained from HWTT may not always predict the moisture susceptibility of the asphalt mixtures. The traditional test parameters extracted from HWTT results, such as total rut depth (TRD) and stripping inflection point (SIP) are not always able to decouple permanent deformation from stripping (Yin et al. 2014; Tsai et al. 2016). Therefore, different analysis approaches were utilized in this study for the evaluation of moisture resistance of Oregon mixes. Tsai et al. (2016) suggested a three-stage Weibull approach to determine the SIP using a fitted HWTD rutting evolution curve, as shown in Figure 4.6.

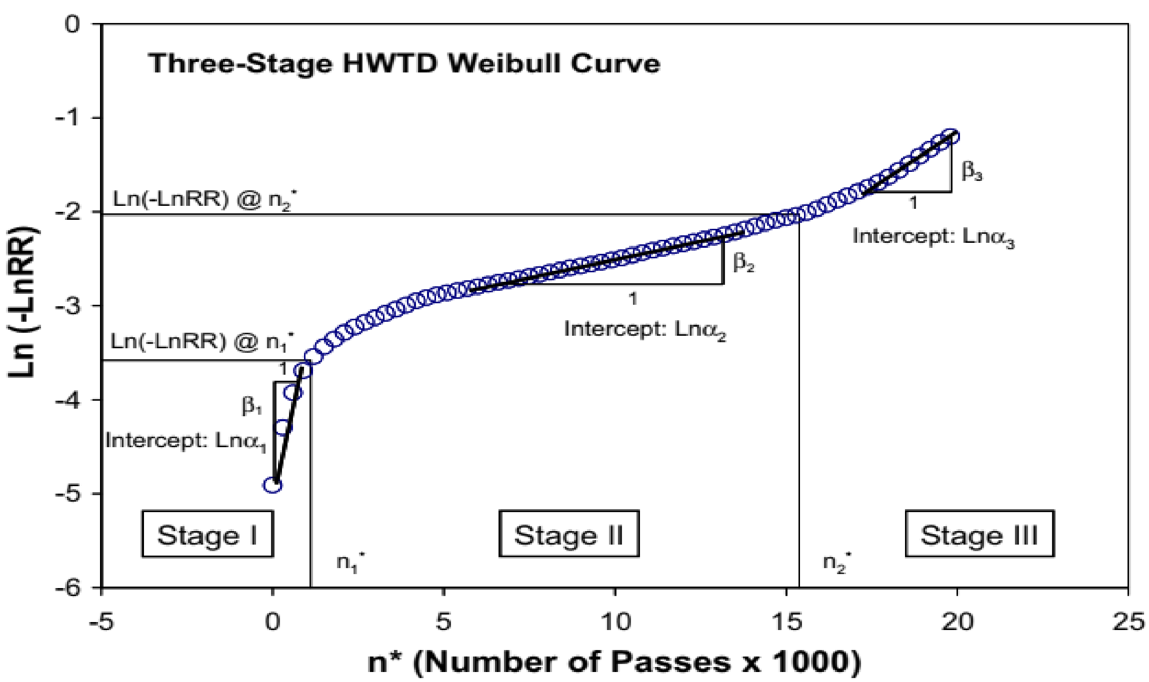


Figure 4.6: Three-stage Weibull curve from HWTT result (Tsai et al. 2016)

Figure 4.6 shows an example of the HWTT Weibull curve with ln(-lnRR) on the y-axis and n* on the x-axis. RR stands for rut-depth ratio, which is defined as the ratio of specimen height (H) minus rut depth (RD) over the specimen height, i.e., RR = (H - RD)/ H, and n* is the number of passes (n) expressed in thousands (n/1000). All parameters shown in Figure 4.6 are derived from mathematical manipulation of the Weibull equation applied to the HWTT Weibull curve. n_2^* is the stripping initiation point that marks the separation between Stage II and Stage III portions of the curve.

Experience from previous research work (Kumar et al. 2021; Coleri et al. 2020) has shown that asphalt mixtures in Oregon generally do not reach Stage III of the rutting accumulation curve. Therefore, rather than the three-stage Weibull approach, a 2-stage Weibull approach developed by Coleri et al. (2008) for the repeated simple shear test at constant height (RSST-CH) data was revised and adapted and used in this study for HWTT data analysis. In this analysis approach, ln(n) versus ln(-ln(PSS)) plots are used to determine the phase separation (n_1) between Stage I and Stage II. PSS stands for permanent shear strain at a given loading repetition. An example of a two-stage RSST-CH Weibull curve is shown in Figure 4.7 below.

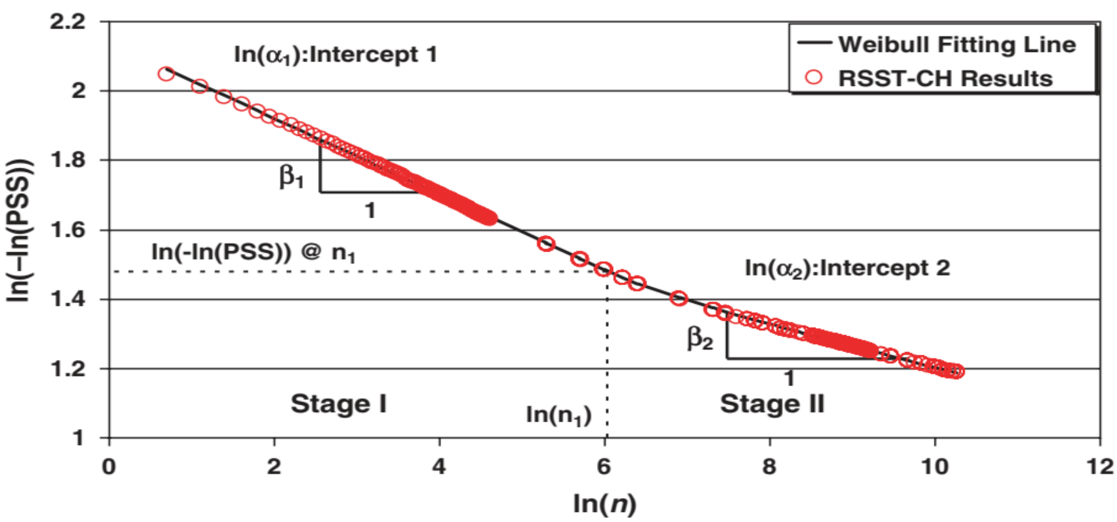


Figure 4.7: Two-stage Weibull curve from RSST-CH result (Coleri et al. 2008)

4.5.4 Boiling Method with Colorimeter

The Boil test method in its original form (ASTM D3625, 2020a and Tex-530-C, 1999) is a visually subjective test carried out on loose asphalt mixture (See Section 2.3.1). A loose asphalt mixture is added to boiling distilled water, and the mix is boiled for 10 minutes. The boiling of the mixture is expected to strip off the asphalt film from the aggregate material. This adhesive failure or stripping is expressed in terms of visible color change relative to the unboiled mixture. The visual observation of the color change of the mixture introduces subjectivity and variability in this test. Therefore, a more quantified approach suggested by Tayebali et al. (2019) was adopted in this study. Tayebali et al. (2019) recommended boiling the mix for 30 mins instead of 10 minutes to reduce variability. Moreover, the subjectivity of the test was eliminated by incorporating a color measuring device. The device used in this study was the colorimeter CM-600D, as shown in Figure 4.8.

Figure 4.8: CM-600D Spectrophotometer (Konica Minolta 2021)

The measurements from CM-600d are in the form of color index, L^* readings, based on grayscale and range from 0 to 100, where 0 indicates the darkest color and 100 the whitest. The test procedure is presented below and also shown in Figure 4.9.

1. Loose asphalt mix specimens were prepared from 2 different buckets of each mix. From each bucket, two replicates weighing 450 g each were collected after splitting the mix in a mechanical splitter (Figure 4.9a).

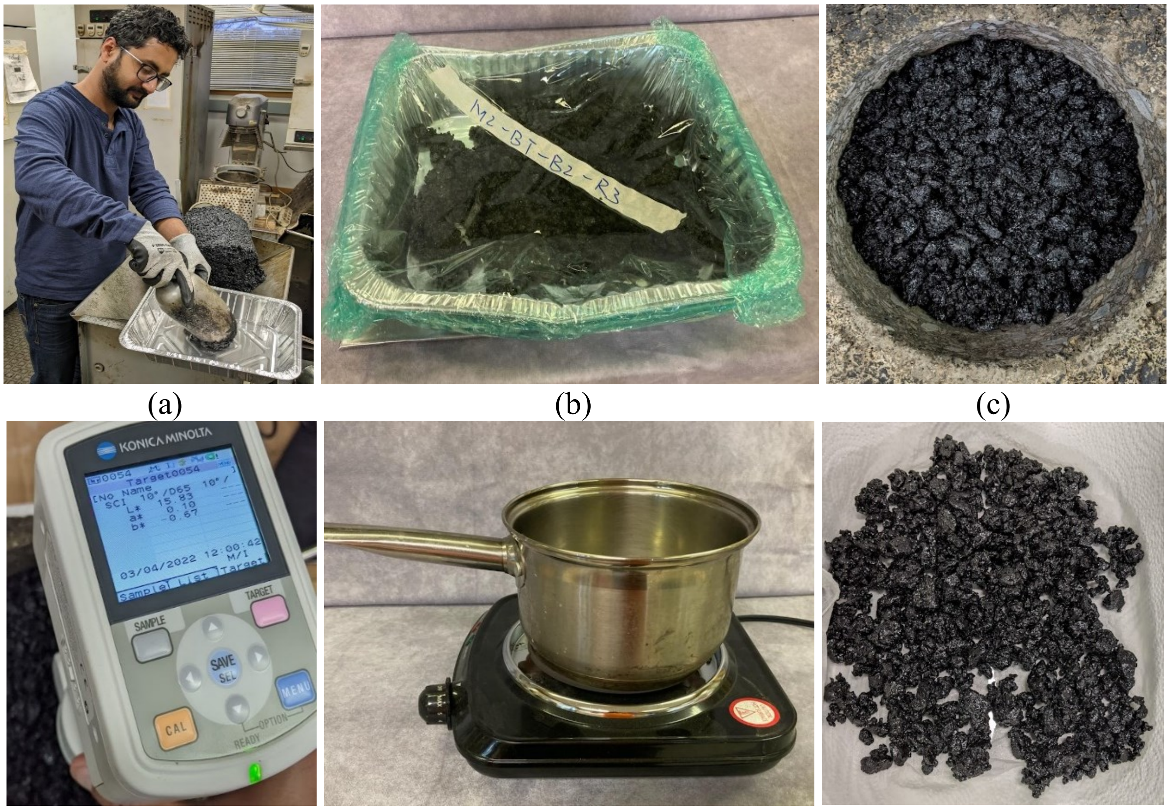
2. The sampled loose mixes were stored in sealed aluminum trays to prevent aging (Figure 4.9b).

3. The loose mix was placed into a cored asphalt block to ensure consistency in the surrounding areas (Figure 4.9c).

4. Ten colorimeter readings were taken randomly (Figure 4.9d) from each control (unboiled) replicates and averaged to calculate the Unboiled L^*. The top surface of the mix was smoothened with a flat metal disc.

5. The dry mixes were boiled in distilled water for 30 mins (Figure 4.9e). Water was decanted after cooling down the wet mix and removing any free asphalt from the water surface. The wet mix was transferred to another aluminum tray on top of a white paper towel. The mix in the sealed tray was allowed to sit for 24 hours.

6. Ten colorimeter readings were taken randomly from the boiled mixture (Boiled L^*) (Figure 4.9f).



(a) (b) (c)

(d) (e) (f)

Figure 4.9: The boil test method with the colorimeter

The average L* values of both boiled and unboiled mixtures were used to calculate the damage ratios or percentage stripping (L^*_{RB}) according to Equation 4-7.

$$\boldsymbol{L^*_{RB}} = \frac{\textbf{Conditioned (boiled) L}* - \textbf{Unconditioned (unboiled) L}*}{\textbf{Unconditioned (unboiled) L}*}\ \boldsymbol{x}\ \mathbf{100}$$

(4-7)

For the broken asphalt cores from the TSR tests, ten readings were taken from each half of the fractured IDT samples. The readings were taken from different locations of the fractured surface such that the whole surface was covered. Instead of boiled and unboiled loose mixtures, here readings from unconditioned (control) and vacuum conditioned samples were used to determine the damage ratio or the percentage stripping. Figure 4.10 shows the fractured IDT samples on which colorimeter readings were taken.

(a) (b)

Figure 4.10: Fractured IDT samples used for colorimeter readings (a) control or unconditioned sample (b) vacuum conditioned sample

4.5.5 Electrical Resistivity Testing

Electrical resistivity is a material property that has been found to be linked with the durability characteristics of concrete (Azarsa and Gupta 2017). Resistivity has also been identified as an index for measuring fluid transport properties. Moreover, electrical resistivity measurement has emerged as a tool for quality control in concrete-related projects (Azarsa and Gupta 2017). Concrete has the presence of dissolved charged ions (pore solution) due to cement chemistry and chemical reactions involved in the process of production and hardening (hydration). For this reason, concrete is electrically conductive, and hence resistivity measurements provide a way to characterize its microstructure. Recent research has shown that the ratio of concrete bulk resistivity to the pore solution resistivity filling the pores in the concrete to be correlated with the permeability of concrete (Azarsa and Gupta 2017). This ratio is known as the formation factor, which originated from petrophysics (Archie 1942) as per Equation 4-8:

$$\mathbf{F} = \frac{\rho_{bulk}}{\rho_{ps}} \qquad \textbf{(4-8)}$$

where:

F = Formation factor

ρ_{bulk} = resistivity of the bulk system

ρ_{ps} = resistivity of the pore solution

The formation factor is also shown to be inversely related to the porosity (Φ) of the system and the pore connectivity (β) within the saturated system by Equation 4-9 (Qiao et al. 2019)

$$\mathbf{F} = \frac{\rho_{bulk}}{\rho_{ps}} = \frac{1}{\Phi\ x\ \beta} \tag{4-9}$$

Asphalt concrete is generally not a good conductor of electricity and does not have any dissolved charged ions like concrete. Moreover, the resistivity and formation factor concepts have not been applied to asphalt to date. Therefore, a trial with only four samples (as a preliminary investigation) was carried out in this study to check whether the procedure used for concrete is replicable to asphalt. This test has the potential to represent and analyze the pore structure (β) of asphalt, which currently is not possible without computed tomography scans of asphalt cores. Consequently, the pore structure can throw light on the pore connectivity and flow paths of water percolating in asphalt pavements, which can provide information about long-term moisture susceptibility.

For this test, the procedure outlined in AASHTO TP 119 (2015) for saturation of concrete cores by ionic solution and measurement of bulk electrical resistivity was adopted. 100 mm tall and 150 mm diameter asphalt mix cores were prepared using SGC. The sample was placed in a vacuum chamber at 6 torrs (or 6 mm Hg) for 3 hours. Subsequently, the sample was vacuum saturated in the vacuum chamber at the same pressure in a calcium hydroxide-saturated, simulated pore solution for another hour. The sample was then removed from the vacuum chamber and was allowed to condition in lime-saturated pore solution for a minimum of 16 hours. After conditioning, the sample was placed between two electrodes with moist sponges covering the top and bottom surfaces of the core for measuring bulk resistivity using a resistivity meter. Since the bulk resistivity device applies AC voltage and measures AC current, the resistance is measured at a frequency when the phase angle is approximately equal to 0 degrees. Resistivity is calculated by multiplying the resistance (R) and a geometry factor (k) using Equation 4-10:

$$\boldsymbol{\rho} = \mathbf{R\ x\ k} \tag{4-10}$$

The geometry factor (k) is the ratio of the cross-sectional area of the core (A) to the length of the sample (L).

In general, the pore solution used in the test is a standard pore solution whose resistivity can be assumed to be 0.127 Ωm for the saturated specimen (Weiss et al. 2020). The pore solution can also be determined using a conductivity probe and reporting the reciprocal of conductivity as the resistivity of the solution. It can be noted that there are several ways of measuring pore solution resistivity (mill charts, thermodynamic modeling, ionic content etc.), but a conductivity probe has been used in this study because of the ease of use. Figure 4.11 shows the vacuum chamber, bulk resistivity meter, and conductivity probe used in the test.

(a) **(b)** **(c)**

Figure 4.11: Electrical resistivity test (a) vacuum chamber (b) bulk resistivity meter, and (c) conductivity probe

4.5.6 Field Moisture Sensors for In-Situ Infiltration Assessment

In-situ infiltration can be measured using the LCS permeameter and the falling head permeameter. The in-situ infiltration capacity is generally evaluated by surface hydraulic conductivity measurements. These test methods essentially measure the discharge time of a known volume of water through the known thickness of the specimen. The hydraulic conductivity is determined using Darcy's law, according to Equation 4-11:

$$\mathbf{K} = \frac{\boldsymbol{aL}}{\boldsymbol{A\Delta t}}\, \boldsymbol{x}\ \boldsymbol{ln}\,\frac{\boldsymbol{h_1}}{\boldsymbol{h_2}} \tag{4-11}$$

where:

K = the hydraulic conductivity

a = area of the permeameter section

A = cross-area of the permeameter section

L = thickness of the specimen

Δt = time interval for drainage of water from h1 to h2

h1, h2 = maximum and minimum hydraulic load above and below the specimen

Another test method gaining popularity, especially in concrete construction (concrete floor slabs), is the in-situ moisture meter. This test method follows ASTM F2170 (2019b). Considering the efficacy and ease of use, this technique has been adopted in this study for detecting moisture infiltration in roller compacted asphalt concrete slabs. The steps involved in this method are described below and also shown in Figure 4.12.

1. Roller compacted specimen of 260 mm x 400 mm x 60 mm (10.2 in x 15.7 in x 2.36 in) was prepared in the laboratory to better represent the pavements constructed in the field.

2. Three holes (sizes) were drilled at three different locations in the slab.

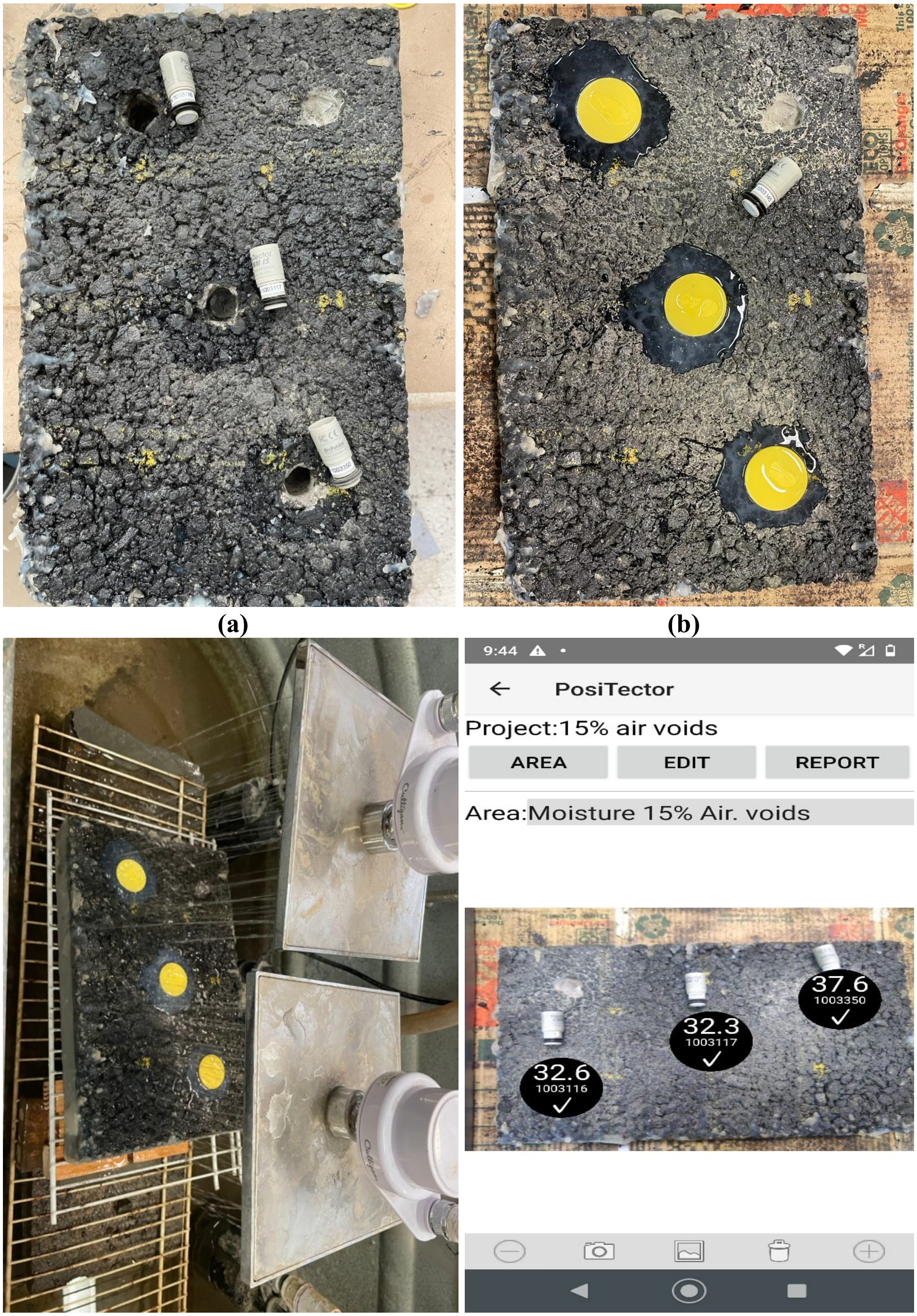
3. The sides of the slab were sealed so as to avoid any lateral infiltration of water. Water was only allowed to seep through the top surface of the slab.

4. Three PosiTector® moisture sensors were inserted in the drilled holes such that the top of the sensors was at the slab surface level. Caps were placed on the sensors, and the circumference of the caps was sealed and flushed with the slab surface to prevent any water from entering through the gap between the caps and the slab surface.

5. The slab with sensors was placed in a rainfall simulator under a constant discharge of water. The slab was tilted at an angle to create water flow only in one direction (with an angle similar to the superelevation on roadways).

6. The sensors connect to any cellphone device via Bluetooth, and the relative humidity (RH) and temperature within the slab are displayed in real-time on an application installed on the cellphone.

Only three sensors were purchased for this study ($400 price), and the cellphone app was used for collecting data. The cellphone app is not capable of logging data automatically. For this reason, data was collected manually by checking the app every 2 to 5 minutes and recording the values. For continuous and automated data collection for longer periods, the version with the data acquisition device needs to be purchased (the price for the complete system with five sensors and the data acquisition device is $1,800). However, for field implementation and monitoring the infiltration of water into the critical locations in the pavement structure, the cellphone app use is expected to be the reasonable method, while the data acquisition device can be useful for longer moisture infiltration monitoring in research studies.



(a) (b) (c) (d)

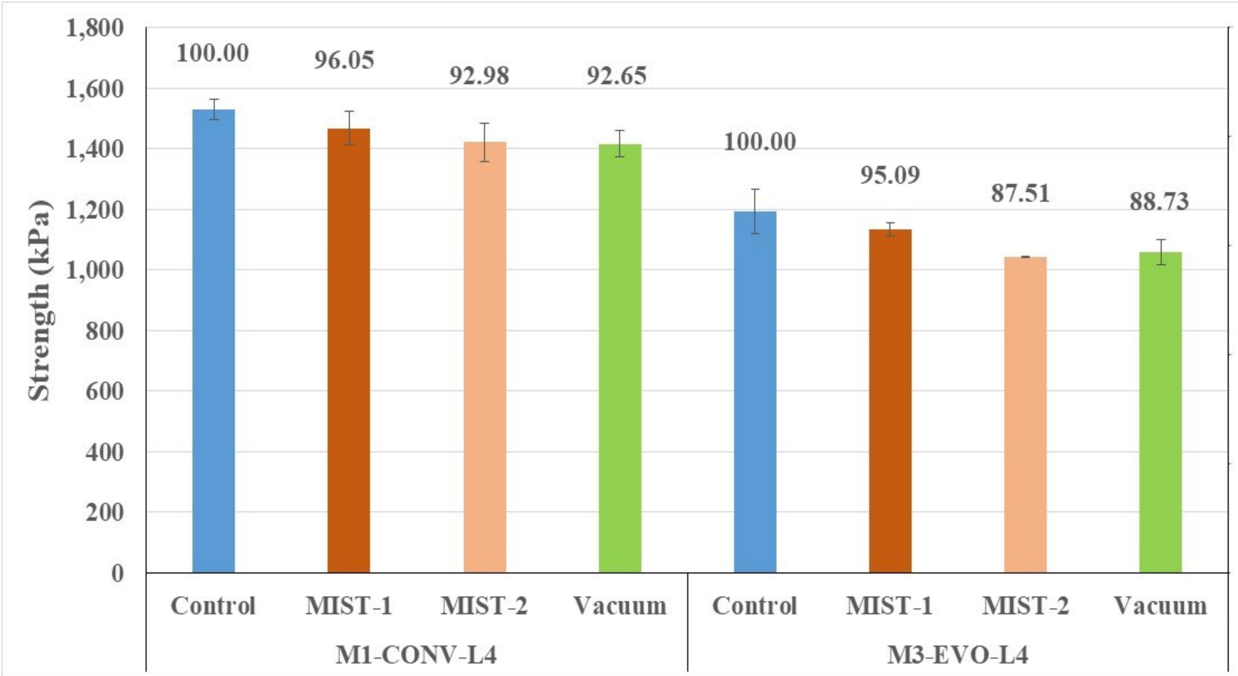
Figure 4.12: In-situ moisture meter test (a) drilled holes and moisture sensors (b) sealed caps of moisture sensors (c) rainfall simulator, and (d) cellphone app

4.6 RESULTS AND DISCUSSION

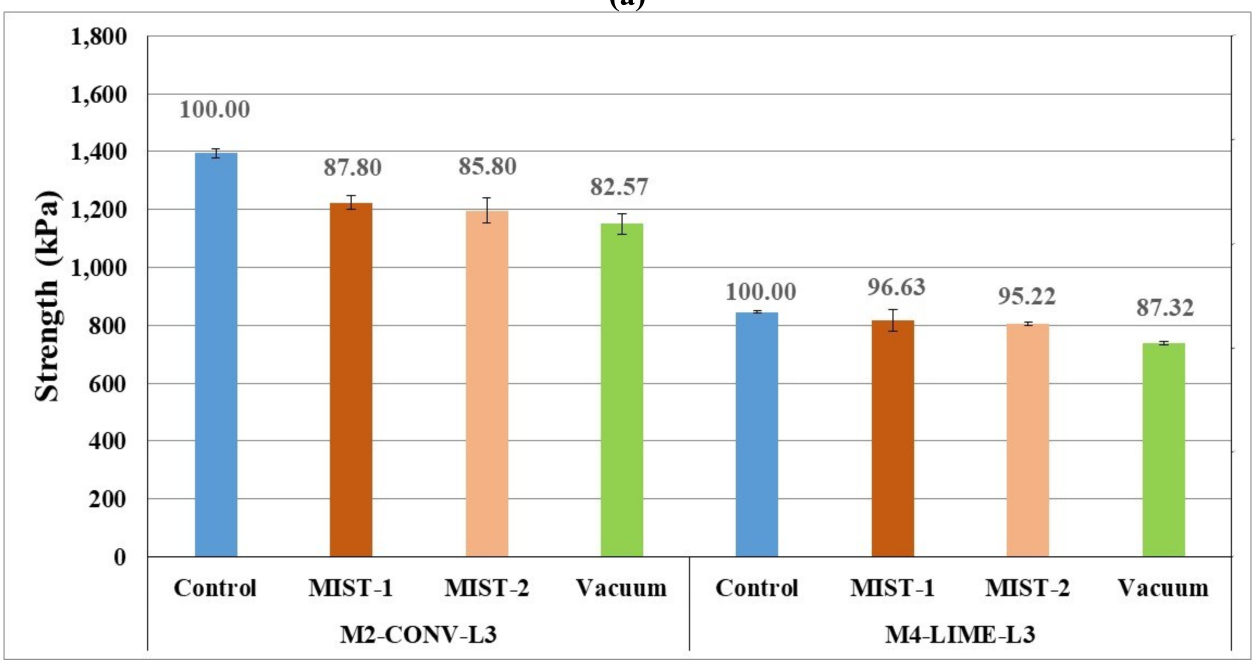
4.6.1 Phase I

4.6.1.1 IDT – TSR (AASHTO T 283, 2014)

Figure 4.13 presents the results of tests for cracking (IDT) performance for different mixtures (conditioned and unconditioned) used in this research project. Strength was calculated from the load-displacement outputs and used to evaluate the cracking performance of all asphalt mixtures. The percentage of strength left due to moisture conditioning (Tensile Strength Ratio -TSR) was also calculated and used as a parameter to evaluate moisture susceptibility. It should be noted that TSR is also the parameter used to evaluate the moisture susceptibility of asphalt mixtures in Oregon (AASHTO T 283, 2014). Three replicate specimens were tested for each conditioning method, and a total of 12 tests were conducted for each project. It should be noted that projects M1 and M3 were Level 4 mixtures (shown in Figure 4.13a) whereas M2 and M4 were Level 3 mixtures (shown in Figure 4.13b). For additional details on the mixtures, please see Table 3.1.



(a)



(b)

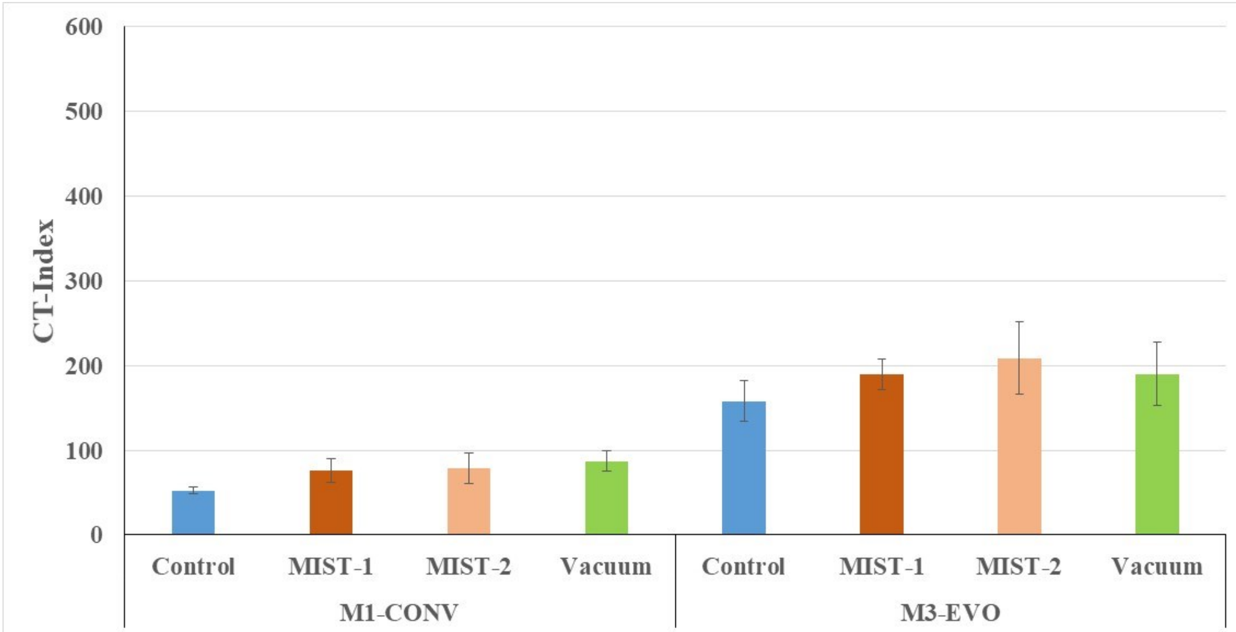
Figure 4.13: Strength and TSR outputs from the IDT test results for M1, M2, M3, and M4 (a) Level 4 mixtures (b) Level 3 mixtures (length of the error bar is equal to one standard deviation, the percentages on every bar are the TSR values)

Note: MIST-1: Moisture conditioning at 50ºC and 40psi; MIST-2: Moisture conditioning at 50ºC and 70psi; Vacuum: Vacuum saturation according to AASHTO T 283 (2014).

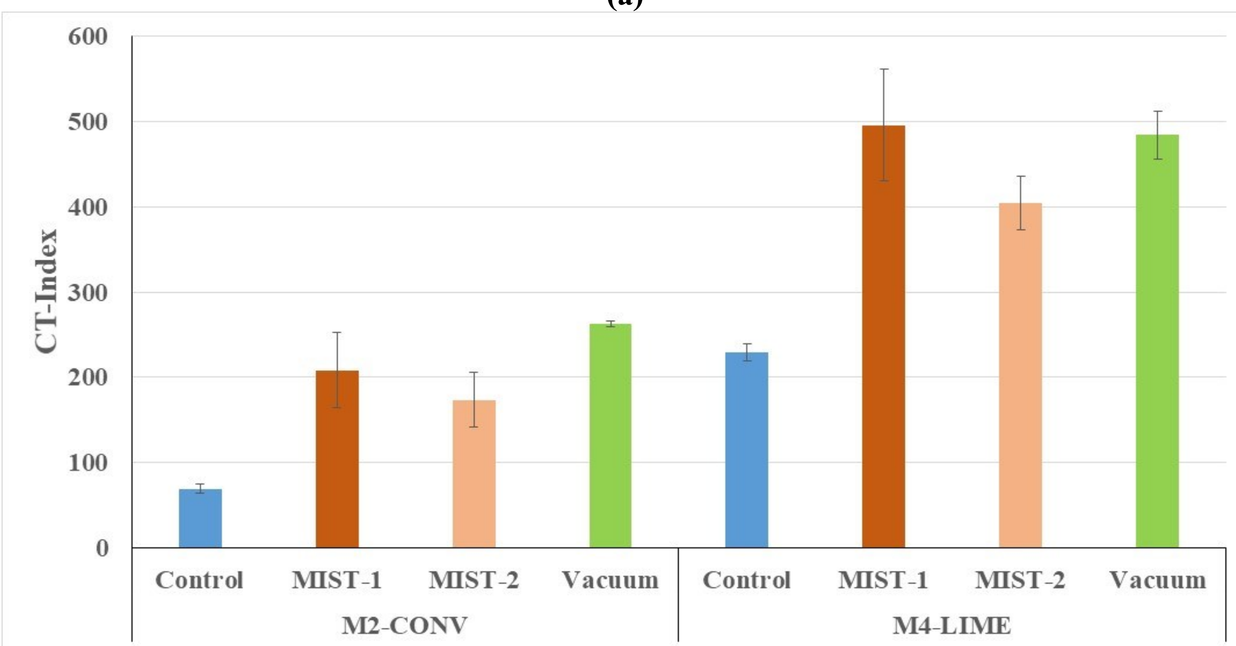
For Level 4 mixtures, it can be observed that the tensile strength of control asphalt mixtures is always higher than the conditioned samples, while the samples conditioned with vacuum saturation have lower strength values than the MIST-1 conditioned samples. The tensile strength of MIST-1 conditioned (50°C, 40 psi) samples is similar to control, indicating that MIST-1 conditioning did not create any significant moisture damage in the samples. On the other hand, MIST-2 conditioning (50°C, 70 psi) resulted in samples with tensile strengths similar to that of the vacuum-conditioned samples for M1 and M3. The reason is that MIST2 conditioning included higher pressure which resulted in more damage to the asphalt concrete microstructure. Additionally, results indicated that MIST-2 conditioned samples had higher strength values than vacuum conditioned samples for all the four mixes except M3, although the difference was not significant. The major difference between M3 and the other three mixes was the warm-mix additive used in M3.

As shown in Figure 4.13, according to the AASHTO T 283 (2014) specification followed by ODOT, both Level 3 and 4 mixtures would perform well in the field as they have retained more than 80 % of their strength after conditioning. This is not particularly surprising as all of the mixes were designed using Superpave mix design specifications, which require the mixture to pass AASHTO T 283 (2014) requirements (needed to retain at least 80% of the strength after conditioning).

Using the same IDT test results, CT-indices were calculated for all test results using a MATLAB code developed at OSU-AMaP. The results are shown in Figure 4.14.



(a)



(b)

Figure 4.14: CT-index outputs from the IDT test results for M1, M2, M3, and M4 (a) Level 4 mixtures (b) Level 3 mixtures (length of the error bar is equal to one standard deviation)

Note: MIST-1: Moisture conditioning at 50ºC and 40psi; MIST-2: Moisture conditioning at 50ºC and 70psi; Vacuum: Vacuum saturation according to AASHTO T 283 (2014).

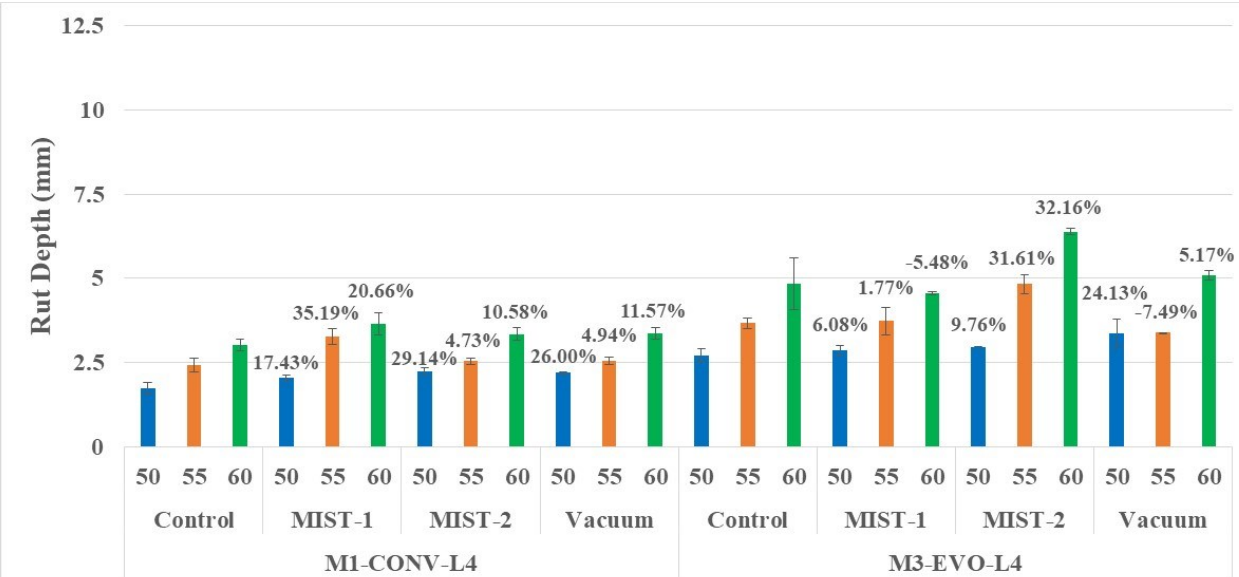
By comparing Figure 4.13 and Figure 4.14, it can be observed that the CT-index values do not agree with the measured strength values. This result is expected since the CT-

Index parameter quantifies the flexibility of the asphalt mixture while strength is highly controlled with the stiffness of the mixture. For instance, M3 with the WMA additive has an average strength value significantly lower than M1, while the average CT-Index value for M3 is about three times higher than the average CT-Index value for M1. For Level 4 mixes, the CT-Index values for all MIST and Vacuum conditioned specimens are similar, while the CT-index values for the Control specimens are significantly smaller than the CT-Indices for the conditioned specimens. Moving to Level 3 mixes, similar to Level 4, the CT-Index values of unconditioned asphalt specimens are always significantly lower than the conditioned specimens for both projects. For all the four mixtures, CT-index values increased as a result of conditioning with either MIST or Vacuum saturation. It can also be observed that CT-Index variability is also significantly higher for the conditioned specimens due to the variable damage created in the asphalt concrete specimen microstructure due to conditioning.

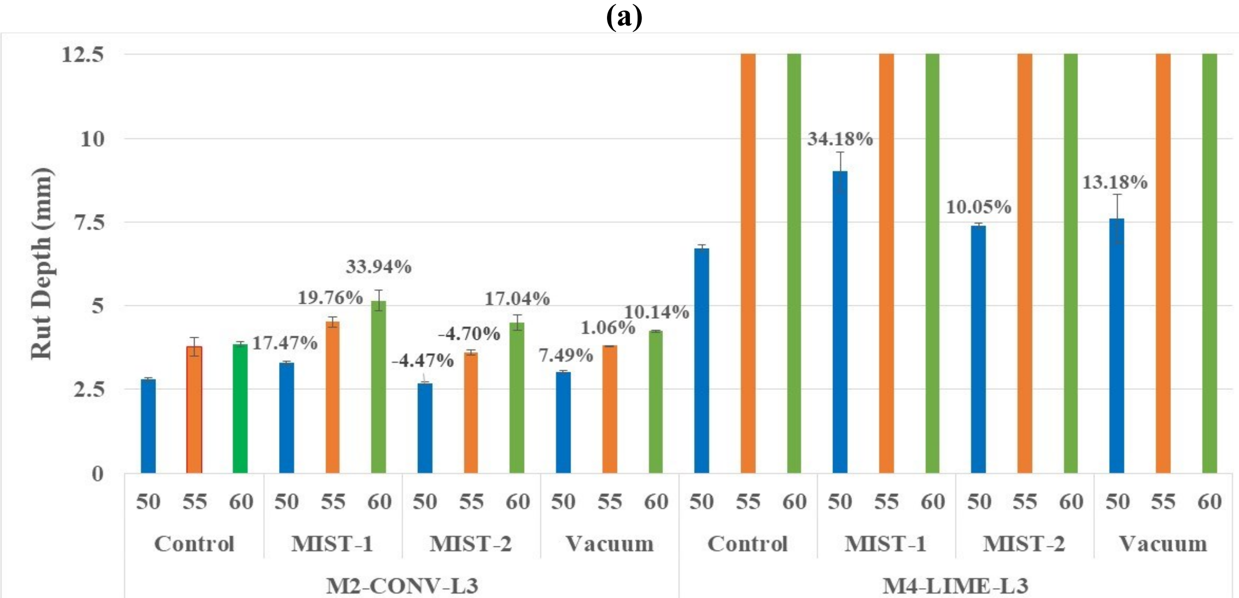
These results suggested that the CT-Index parameters calculated from IDT test results might not be capturing the moisture susceptibility of the asphalt mixtures, while the strength parameter (which is also the parameter currently used for moisture susceptibility evaluation in AASHTO T 283, 2014) can provide a more reasonable output correlated with moisture susceptibility. The underlying assumption in the fracture tests (for both IDT and SCB) is that the entire fracture energy is dissipated through actual material failure through cracking. However, in reality, a small portion of the energy is also dissipated for creep and plastic deformation accumulation in the mix when a specimen without any conditioning was tested. Since conditioning creates damage in the specimens, conditioned specimens become more prone to creep and plastic deformation. Due to the additional energy required to overcome creep and plastic deformation resistance, more energy is required to create failure in the conditioned mixtures. This phenomenon is expected to be the reason for higher CT-Indices for conditioned specimens in this study. The CT-Index differences between control (unconditioned) and conditioned specimens are higher for Level 3 mixes since they have higher binder contents and softer binder types, which ultimately resulted in higher creep and plastic deformation in the test. In a recent research study, Dave et al. (2018) obtained similar results when the unconditioned and conditioned specimens were tested with SCB, and the flexibility index (FI) (another test parameter that is similar to the CT-Index defining the flexibility of the mixture) parameter was used for evaluation. Thus, in this study, CT-Index or FI parameters that are related to the flexibility and cracking resistance of the asphalt mixtures are not recommended to be used to evaluate the moisture susceptibility of asphalt mixtures.

4.6.1.2 HWTT Results

The results from Hamburg wheel tracking tests (HWTT) performed according to AASHTO T 324 (2019b) in Phase I of this study are shown in Figure 4.15. All samples were tested in the water while the water temperature controlled the specimen temperatures. Control results are for the samples that were not subjected to any moisture conditioning before testing. The numbers at the top of each bar show the percentage difference in rutting depth between the control and conditioned samples. The gyratory cores tested in the HWTD were subjected to standard 20,000 passes submerged in water at temperatures 50ºC, 55ºC, and 60ºC. Since some of the specimens for M4 exceeded the 12.5mm rut depth at high temperatures before reaching the 20,000 load repetitions, those results were shown as ending at 12.5mm in Figure 4.15 to make the other results more visible.



(a)



(b)

Figure 4.15: HWTD test results for different conditioning methods (a) Level 4 mixtures (b) Level 3 mixtures (length of the error bar is equal to one standard deviation, the numbers at the top of each bar show the percentage difference in rutting depth between the control and conditioned samples)

Note: MIST-1: Moisture conditioning at 50ºC and 40psi; MIST-2: Moisture conditioning at 50ºC and 70psi; Vacuum: Vacuum saturation according to AASHTO T 283 (2014).

Figure 4.15(a) shows the rut depths for the Level 4 mixes, and Figure 4.15(b) shows the HWTT results of the Level 3 mixes. It can be observed from Figure 4.15 that increasing the test temperature increases the mixture rutting (as expected). In general, rut depths for the conditioned specimens are higher than the control specimens due to the damage created in the specimen microstructure as a result of the conditioning.

For test results for the M4 mix (the mix treated with hydrated lime), all the samples tested at temperatures higher than 50ºC irrespective of the conditioning method, exceeded the 12.5 mm rut depth threshold. Both M2 and M4 contained softer binder grades (PG 64-22 and PG 64-28, respectively), with M4 containing higher binder content (6.3%) with a higher VFA (75%) when compared to the binder content of the M2 mix (5.7%) with a significantly lower VFA (71%). The higher rut depth values for M4 samples point to the effect of higher binder content and VFA on rut resistance. M4 asphalt mixture is expected to fail from rutting if the pavement temperatures around that roadway section exceed 55ºC for a significant portion of the year.

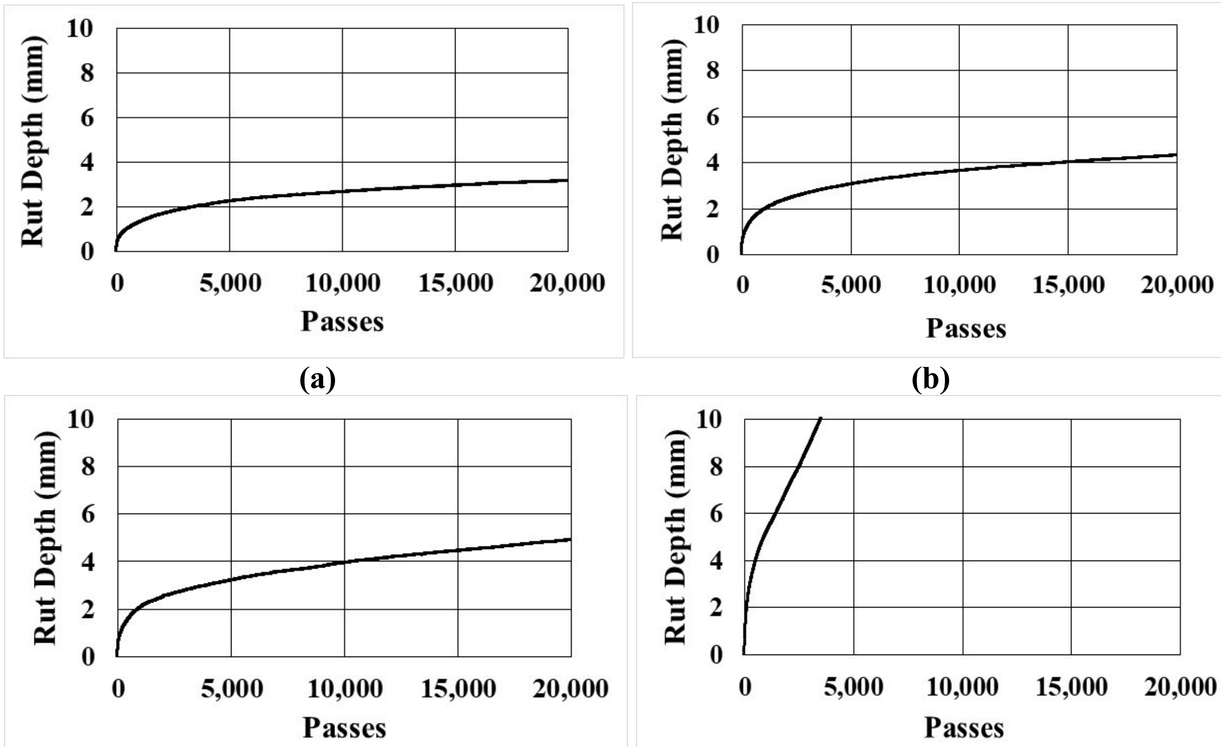
It can be observed that the percentage increase in rut depths at 50ºC due to conditioning is significantly higher for the M4 mix when compared to the M2 mix. This result contradicts the TSR results for M2 and M4 presented in Figure 4.13(b) (in which M4 had a higher TSR value). Although it is not possible to identify which test is providing a better estimate of moisture susceptibility due to the lack of any field moisture resistance/performance data, the M4 mix is expected to have a higher moisture resistance when compared to M2 because it had lime in it. The significant change in rut depth for M4 due to conditioning might be a result of the higher binder content of the M4 mix. Since the test results for HWTT are controlled by both rut resistance and moisture susceptibility, higher rutting accumulation for M4 might be dominating the test results and masking the impact of moisture susceptibility on test results. This issue with using HWTT results for moisture susceptibility evaluation is also discussed in the literature review section with examples from several research studies (See Section 2.3.8).

It is also important to note that M4 had an accumulated average total rut depth of 6.7mm after 20,000 repetitions for the specimens that were not exposed to any kind of moisture conditioning. Although this level of rutting is below any HWTT rut depth thresholds implemented for balanced mix design by several state DOTs (a common threshold is 12.5mm), increasing the temperature for just 5ºC resulted in an average rut depth of about 26mm (a number obtained by linear extrapolation using the HWTT results). This result suggests that it might be necessary to select a higher testing temperature for the mixes that will be used for construction in areas that are expected to experience higher summer temperatures to avoid any rutting failures. A potential modification for HWTT test temperatures for different areas in Oregon, rather than just using the standard 50ºC, will be investigated in future research.

Level 4 mixes showed similar increases in rut depths as Level 3 mixes with increasing temperature for all conditioning methods. For both M1 and M3 samples, the observed rut depths were smaller than the Level 3 mix rut depths. It is also important to note that none of the Level 4 mix samples reached or got closer to the 12.5 mm rut depth in any conditioning method. The reason behind this can be the use of polymer-modified asphalt, lower binder content (as a result of the higher number of gyrations required for designing Level 4 mixes), and stiffer binder grade in the Level 4 mixes (M1 and M3). The percent difference in rut depths given at the top of each bar shows the difference in rut depth between the control samples and the conditioned samples. Based on the calculated percentages for the 50ºC test results, M1 and M3 are expected to have similar moisture susceptibility (26% and 24.13% increase in rut depth, respectively). This result also does not agree with the TSR results. According to the TSR results, M1 has about 4% higher TSR value than M3. However, it should be noted that M3 has a WMA additive which is expected to increase the moisture resistance of the mix and provide a TSR that is higher than M1.

4.6.1.3 Two-stage Weibull analysis

As discussed in Section 2.3.8, the rut depth at the end of specific load repetition may not always capture the moisture damage susceptibility of a mix. Moreover, the HWTT results and the rutting accumulation curves of the samples tested in this phase revealed that none of the mixes experienced stripping damage at the standard test temperature of 50ºC. At 55ºC and 60ºC, only M4 showed the tertiary flow that is needed to calculate the stripping inflection point (SIP) and stripping inflection slope parameters (which is generally a result of the reduced rut resistance due to increased test temperatures and not due to high moisture susceptibility). As also discussed in Section 2.3.8, SIP and the stripping inflection slope are not considered to be effective parameters for moisture susceptibility evaluation by many studies since those parameters are highly controlled by the rut resistance of the mixture (See Section 2.0). Since the majority of the asphalt mixtures used for the construction of roadways managed by ODOT have high rut resistance, it is unlikely to observe the tertiary flow required to calculate the moisture susceptibility-related parameters (such as SIP). Figure 4.16 shows four representative HWTT plots from the four mixes.



(a) (b)

(c) (d)

Figure 4.16: HWTT rutting accumulation curves (a) Mix 1-60°C-Vacuum conditioned (b) Mix 2-60°C-Vacuum conditioned (c) Mix 3-60°C-Vacuum conditioned (d) Mix 4-60°C-Vacuum conditioned

It can be observed from Figure 4.16 that only Mix 4 [Figure 4.16(d)] plot has the presence of the tertiary stage (or stripping phase) of rut accumulation at 60°C (10°C higher than the standard HWTT temperature of 50°C). For all other mixes, the curve propagates only up to the secondary stage. It should also be noted that the plots shown in Figure 4.16 are for the vacuum-conditioned HWTT samples that exhibited the maximum rut depth among all conditioning methods (see Figure 4.15). Thus, these plots represent the 'worst' scenario. Since the stripping phase was absent for all four mixtures at the standard 50°C test temperature, a two-stage Weibull analysis, developed by Tsai et al. (2016), was performed on the test results as described in Section 4.5.3. The Weibull parameters B_1 and B_2 were determined from the analyses. B_1 and B_2 indicate the slopes of Stage I and Stage II Weibull curves, respectively (See Figure 4.7). In this study, the B_1/B_2 ratio is used to evaluate the moisture susceptibility of the asphalt mixtures. This parameter was selected based on the findings from the results of the HWTD tests conducted with asphalt specimens submerged in water and dry (See Section 4.6.4.1 for more details). The change in slope during the test is expected to be a result of the moisture resistance of the asphalt mixture (smaller ratios meaning higher resistance to moisture). HWTT results conducted with Mix 2-Control specimens submerged in water and dry are shown in Figure 4.17. It can be observed that testing the specimen in water results in a faster deformation accumulation at the first stage, while the trend of the curves after the initial stage are similar. This trend was repeatable for all evaluated asphalt mixtures, and more details are provided in Section 4.6.4.1. In Phase I, HWTD tests were conducted with "Control" (meaning no conditioning), MIST-1 conditioned, MIST-2 conditioned, and Vacuum conditioned samples and the B_1/B_2 ratios calculated from the test results using the Weibull algorithm are shown in Figure 4.18.

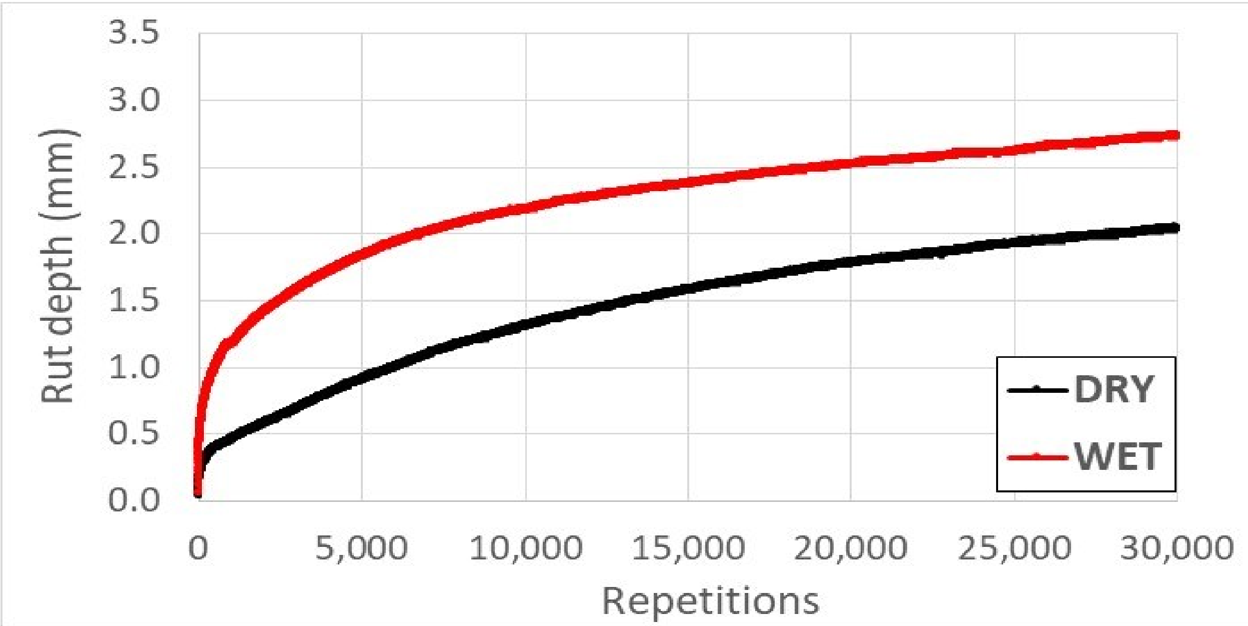


Figure 4.17: HWTT results for Mix2-Control specimens tested with specimens submerged in water and dry

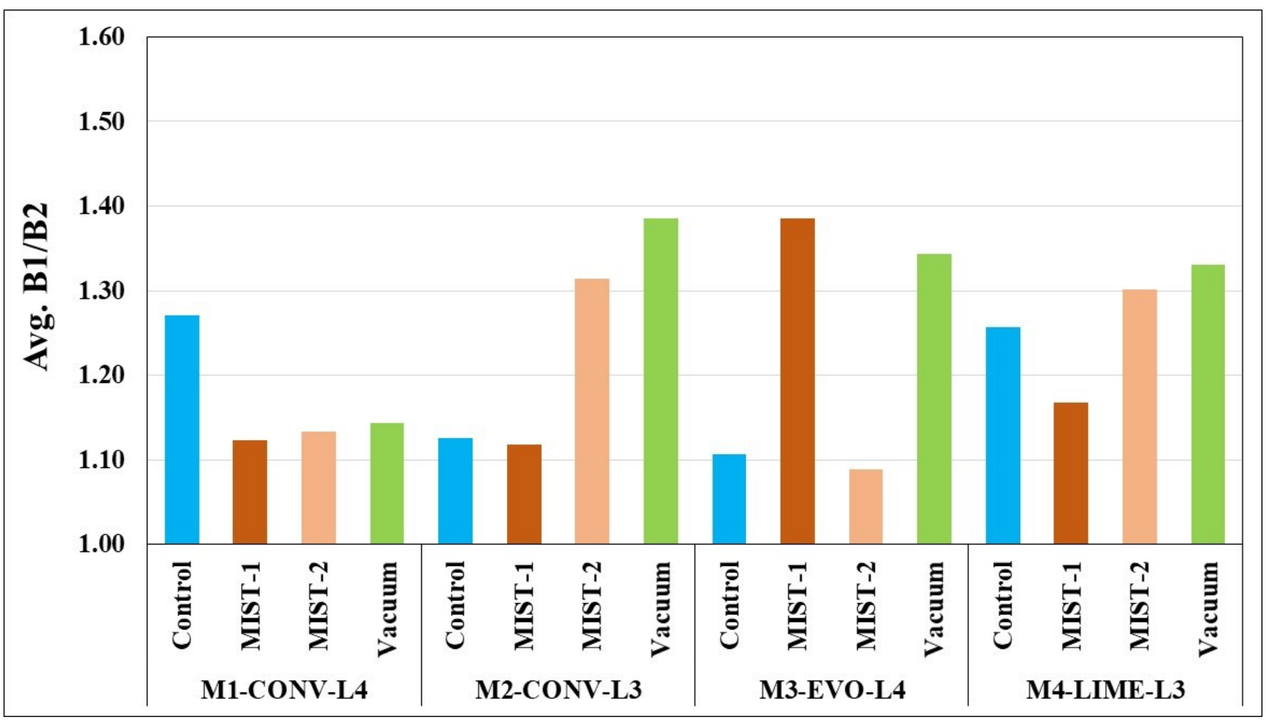


Figure 4.18: B1/B2 Weibull parameter for all conditioning methods for mixes 1 to 4

Results show that the B1/B2 parameter is not able to separate the mixtures with additives from the conventional ones. Specimen conditioning methods also appear to significantly change the measured responses. For this reason, it is not possible to derive reliable conclusions regarding the moisture susceptibility of the asphalt mixtures from the B1/B2 ratio. However, additional tests were conducted for the mixes 5 to 10, and the results from Phase I and II were compiled to get a clearer picture regarding the effectiveness of this parameter for moisture susceptibility evaluation of asphalt mixtures. These results are shown in Section 4.6.2.3.

As discussed in the literature review section (Section 2.0), HWTT is assumed to simulate the vehicular loading and moisture damage accumulation in the field by getting the water into the asphalt concrete pores and creating high pore pressures in the void space with the applied test loads. To determine the level of moisture absorption created in the HWTT system, the weights of the tested cores were measured before and after the HWTD tests (called Unconditioned). Core weights were also measured after MIST1, MIST2, and Vacuum saturation and after HWTD testing following those three saturation methods. Figure 4.19, Figure 4.20, and Figure 4.21 show the percentage of water absorption by dry asphalt core weight for Mix 2, Mix 3, and Mix 4, respectively. It should be noted that measured weights and overall results were highly affected by: i) the water escaped from the core while moving it out of the HWTD molds; ii) small specimen particles that were removed during the HWTD tests and specimen removal; and iii) the variable effort to remove the excess water around the cores to reach the saturated surface dry (SSD) condition before weighing. However, since all the specimens were exposed to the same process, the results can be accepted to be comparable.

Results show that specimens conditioned with MIST or Vacuum had significantly higher water absorption than the unconditioned specimens (specimens tested in the HWTD while submerged in water) at the end of the HWTD tests. This result indicates that HWTD is not accurately simulating a high saturation level during the tests, which might result in minimal pore pressures under vehicular loads that do not represent the actual field conditions in areas with heavy and frequent rain events (such as Oregon). For this reason, Vacuum or MIST conditioned specimens might be required for HWTD testing to achieve a high specimen saturation level (and internal microstructure damage) that simulates the actual asphalt layer saturation during the rainy seasons in Oregon. If a significant portion of the void space in the asphalt layers is not filled with water, vehicular loads may never create high critical stress levels that can damage the material internally.

According to Figure 4.19, Figure 4.20, and Figure 4.21, vacuum saturation resulted in a higher saturation level than MIST in almost all cases. However, the difference was not high. It can also be observed that since MIST and Vacuum saturation allowed reaching a high saturation level in the specimens, the increase in saturation during the HWTD tests was not high (for Mix 4, almost no change). This result shows that MIST and Vacuum saturated specimens were highly saturated during the entire HWTD test duration. Specimens tested in HWTD without any prior conditioning experienced a significantly lower saturation level at the beginning of the experiments.

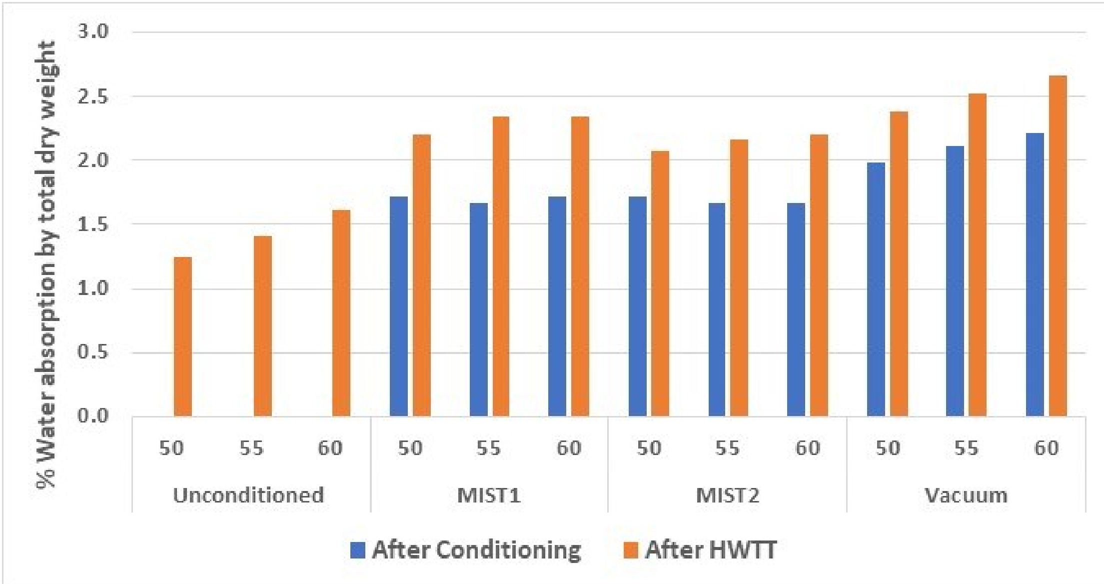


Figure 4.19: Percentage water absorption by dry asphalt core weight at different temperatures (^oC) for Mix2

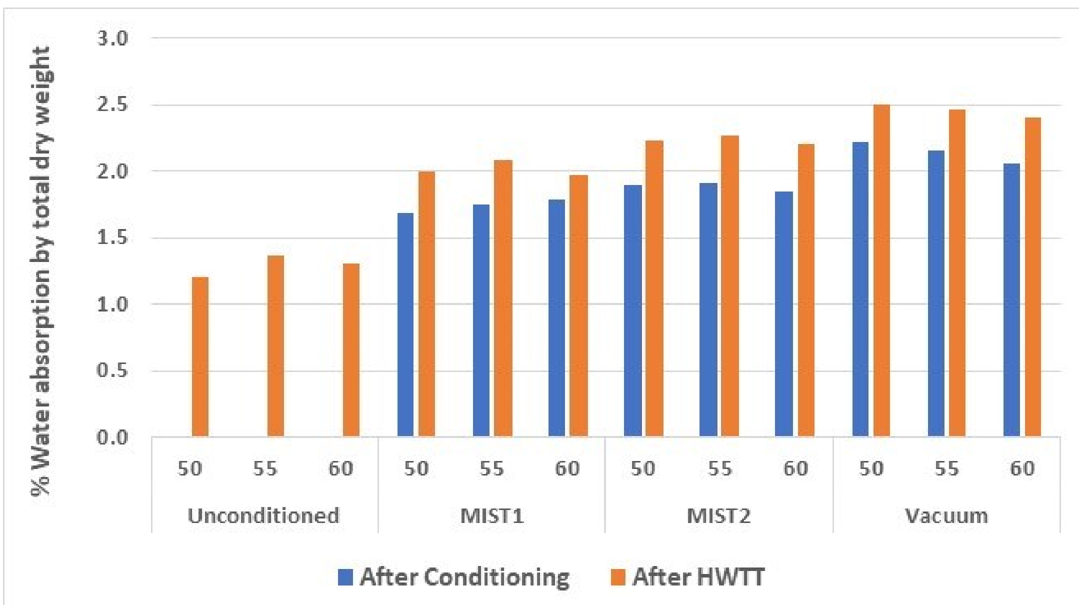


Figure 4.20: Percentage water absorption by dry asphalt core weight at different temperatures (^oC) for Mix3

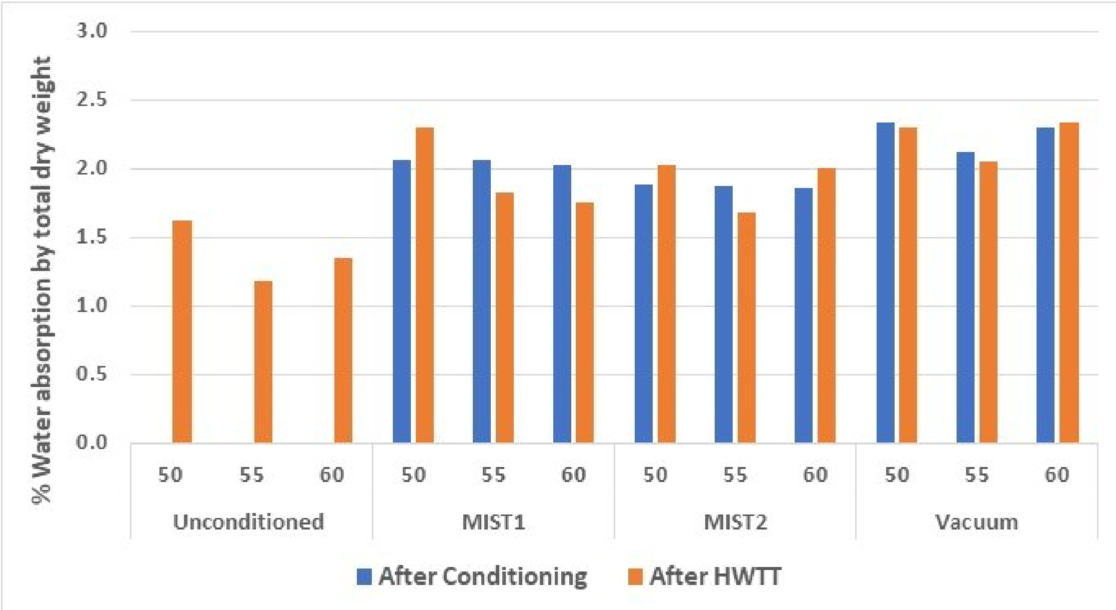


Figure 4.21: Percentage water absorption by dry asphalt core weight at different temperatures (°C) for Mix4

4.6.2 Phase II

4.6.2.1 IDT – TSR (AASHTO T 283, 2014)

IDT strengths for all Phase II asphalt mixes are presented in Figure 4.22. The bars represent the average strength from three replicate experiments. The error bar on every colored bar shows the variability of three replicate tests. The length of the error bar is equal to one standard deviation. Overall, the M10 mix had the highest strength among all the mixes for cases with and without conditioning, while M8 had the lowest strength among all the mixes for cases with and without conditioning. It should be noted that since strength is not directly correlated with the flexibility (or ductility), the long-term cracking performance of the evaluated mixes cannot be ranked based on the measured strength values. Unconditioned samples showed higher strength for all mixes.

It should be noted that these tests complement the tests conducted in Phase I and are presented in Figure 4.13. However, the tests for the specimens conditioned with MIST1 (40psi and 50°C) were not included in Phase II since the low water pressure level of MIST1 was not causing significant damage to the asphalt specimens, according to the findings from Phase I (see Section 4.6.1.1).

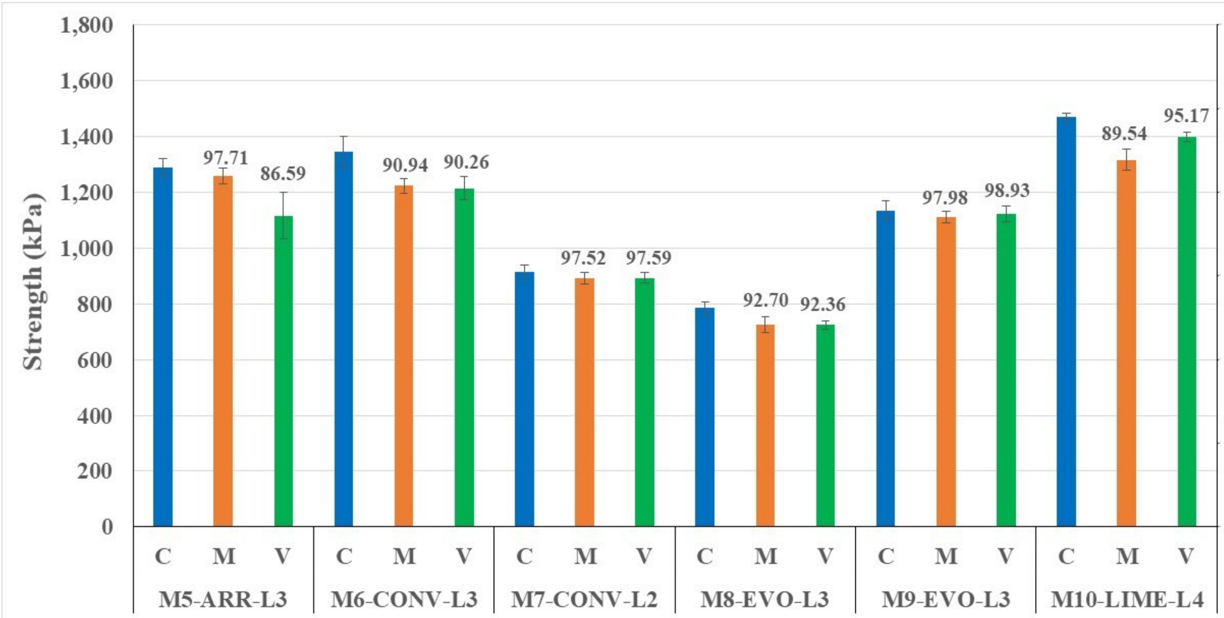


Figure 4.22: Strength and TSR outputs from the IDT test results for mixes M5 to M10 (length of the error bar is equal to one standard deviation, the percentages on every bar are the TSR values)

Note: C: Control-No moisture conditioning; M: MIST-2, Moisture conditioning at 50^{o}C and 70psi; V: Vacuum saturation according to AASHTO T 283 (2014).

As shown in Figure 4.13 and Figure 4.22, all Level 3 and Level 4 mixtures are expected to have high moisture resistance in the field as they have retained more than 80 % of their strength during vacuum and MIST2 conditioning. However, the expectation was to have M3 (Evotherm modified with 0.5% content), M4 (lime modified), M5 (with ArrMaz antistrip), M8 (Evotherm modified with 0.36% content), and M9 (Evotherm modified with 0.5% content), and M10 (lime modified) mixtures to have higher moisture resistance due to the presence of modifiers in their constituents. However, there are no obvious differences in measured TSR (Vacuum) values for those mixes when compared to M1, M2, M6, and M7, which did not have any additives to improve their moisture resistance. Similar results were also obtained when the results for MIST2 conditioned specimens were evaluated. However, it should be noted that the moisture susceptibility rankings for Vacuum and MIST2 saturated specimens were different, while the results were also not highly correlated (only a 26% correlation was achieved). The TSR rankings (in ascending order-worst to best) for both conditioning methods are given as follows:

MIST2 ranking:

M2 M3 M10 M6 M8 M1 M4 M7 M5 M9

Vacuum (T 283) ranking:

M2 M5 M4 M3 M6 M8 M1 M10 M7 M9

Figure 4.23 shows the impact of lime and Evotherm use on measured TSR (AASHTO T 283, 2014) values. Although the average TSR values are higher for the mixtures with lime and Evotherm, the impact of additive usage on TSR values is not high. However, it should be noted that additives are not always the only (or major) factor controlling the moisture susceptibility of asphalt mixtures. Other mixture variables (such as asphalt binder content, RAP content, binder type, VMA, VFA, etc.) also control moisture resistance. It is possible that the impact of some of those major mixture variables might be masking the impact of those additives on TSR and moisture resistance. For this reason, a statistical analysis method (Analysis of Variance (ANOVA)) was used to determine the major factors controlling the TSR values for Vacuum and MIST2 saturated specimens. It should be noted that since the mixture variables were not uniformly distributed (seven mixes had 30% RAP, one had 28%, one had 25%, and one had 20% RAP) and the sample size was limited to 8 (excluding M1 which did not have a measured effective binder content and D/B ratio and M2 which did not have a measured Dust-to-Binder ratio), results of ANOVA may not be highly accurate. However, ANOVA results are still expected to indicate the relative impact of different factors on measured TSR values. These results are also expected to explain the effectiveness of TSR (with MIST2 and vacuum conditioning) in identifying the impact of several mixture variables on moisture susceptibility.

Table 4.7 and Table 4.8 show the results of the analysis for TSR-Vacuum (AASHTO T 283, 2014) and TSR-MIST2, respectively. A "Pr(F) value" lower than 0.05 indicates that the independent variable (any of the mixture variables) is controlling the dependent variable (measured TSR values) with a high level of statistical significance. A higher "F value" means that that particular parameter is significantly controlling the dependent variable (the higher the F value, the higher the impact on TSR). Table 4.7 shows that all Pr(F) values for all mixture variables are smaller than 0.05. This result suggested that all mixture variables are directly controlling the measured TSR-Vacuum test results. With a significantly higher F value (45,568), RAP content is by far the most significant variable controlling the TSR-Vacuum (AASHTO T 283, 2014) test results. To clearly show the impact of RAP on TSR-Vacuum results, Figure 4.24 was developed. It can be observed that increasing RAP content results in lower TSR values and potentially lower moisture resistance.

Total binder content and the effective binder content are also significant parameters controlling the TSR-Vacuum values with high F values. Dust-to-binder ratio was determined to have the least impact on the TSR-Vacuum values (although still statistically significant), while the additive type (no additive, lime, chemical additive) was determined to have a higher impact. These results suggested that TSR-Vacuum results reflect the impact of mixture variables on performance. In addition, significantly higher F values for RAP and binder content variables point out the possibility of those mixture variables masking the impact of additives on TSR-Vacuum values and, ultimately, moisture susceptibility. This result also suggested that it is not possible to achieve high moisture resistance by only using lime or a chemical additive. Selecting the correct mixture variables is also important for reducing moisture susceptibility.

It can be observed from Table 4.8 that none of the mixture variables are affecting the TSR-MIST2 values. This result points out that MIST2 conditioning may not be providing TSR test results that reflect the impact of different mixture properties on the moisture susceptibility. This conclusion might be a result of the aggressive moisture conditioning resulting in highly variable damage in the asphalt mixture microstructure.

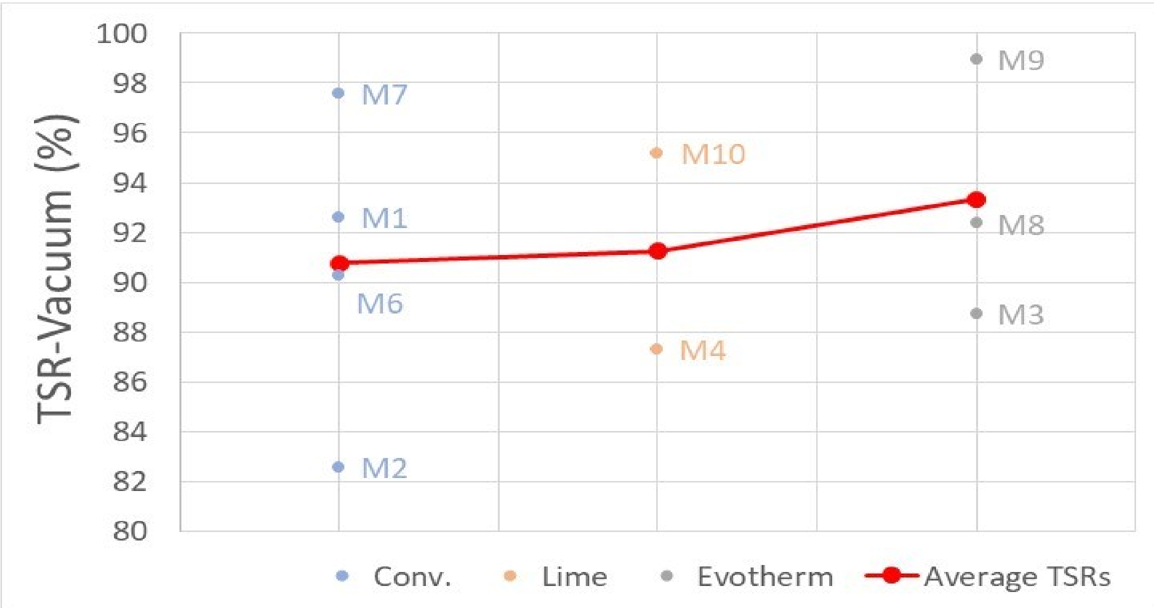


Figure 4.23: Impact of lime and Evotherm use on TSR-Vacuum results

Table 4.7: ANOVA Table to Determine the Mixture Variables Controlling TSR-Vacuum Results, Including RAP Content, Binder Content, Effective Binder Content, Dust-To-Binder Ratio, And Additives

	Sum of Squares	F Value	Pr(F) Value
RAP	97.82	45,568	0.003
AC	19.47	9,071	0.007
P_{be}	16.02	7,462	0.007
D/B ratio	1.65	770	0.023
Additive	16.03	3,733	0.012

Table 4.8. ANOVA Table to Determine the Mixture Variables Controlling TSR-MIST2 Results, Including RAP Content, Binder Content, Effective Binder Content, Dust-To-Binder Ratio, And Additives

	Sum of Squares	F Value	Pr(F) Value
RAP	39.95	0.92	0.513
AC	25.38	0.59	0.584
P_{be}	0.64	0.01	0.923
D/B ratio	6.05	0.14	0.772
Additive	0.19	0.00	0.998

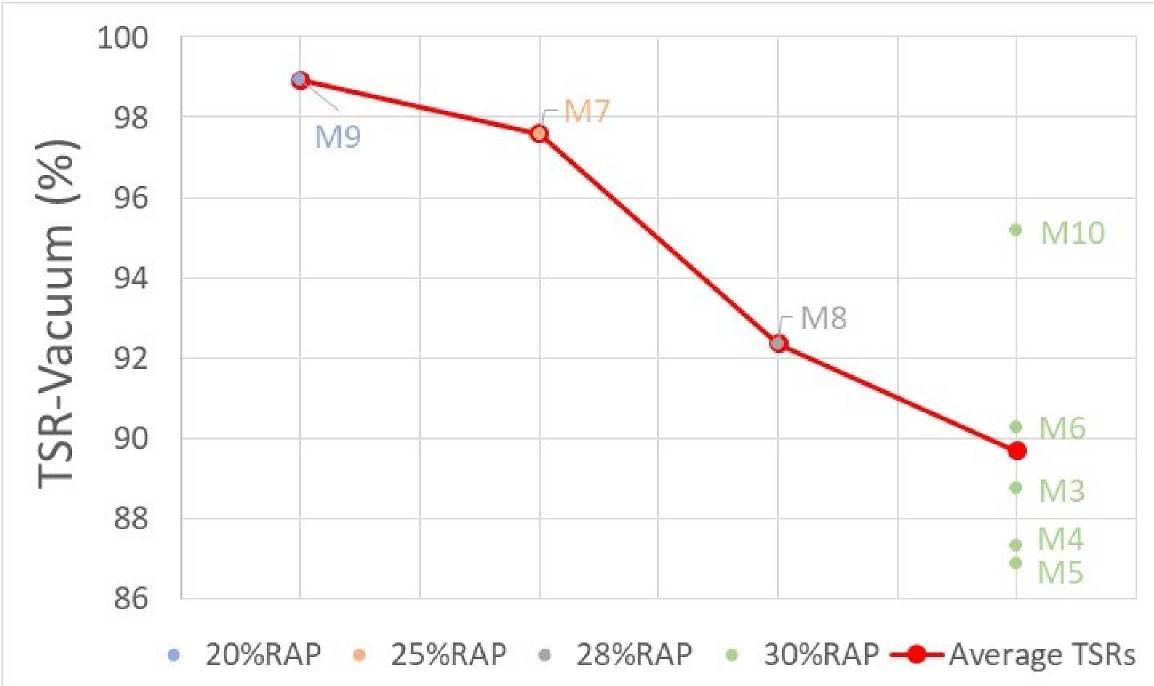


Figure 4.24: Impact of RAP content on TSR-Vacuum results

M5 and M6 were produced at the same plant and had the exact same ingredients except the additional antistripping (ARR) additive used in M5. Due to the presence of liquid antistrip in M5, it was expected to have a higher TSR value (meaning higher moisture resistance). However, according to the TSR values for vacuum conditioned M5 and M6 samples, M6 had a higher TSR value. All these results suggested that the AASHTO T 283 (2014) specification may not accurately identify the mixes with high and low moisture susceptibility. However, this discrepancy might also be a result of the variability observed during the plant mixture production. In SPR826 (Kumar et al. 2021), mixing antistrips at high production temperatures was also determined to result in a reduction in cracking resistance and losing the benefits that can be created by the additive. A similar issue related to the high mixing temperatures might be eliminating the positive expected impact of antistrip additive on moisture susceptibility. Production binder content variability might be another reason.

Figure 4.25 shows the CT-index values calculated from the IDT test results. It can be observed that vacuum conditioned samples have the highest CT-index values, followed by MIST2 conditioned samples, while the unconditioned samples showed the lowest CT-index of the mixes. These results indicate that the conditioning of the samples either in the MIST2 or vacuum chamber improved the CT-index of the samples. These results agree with the findings presented in Section 4.6.1.1 that CT-Index is not a reliable parameter to quantify the moisture susceptibility of asphalt mixtures.

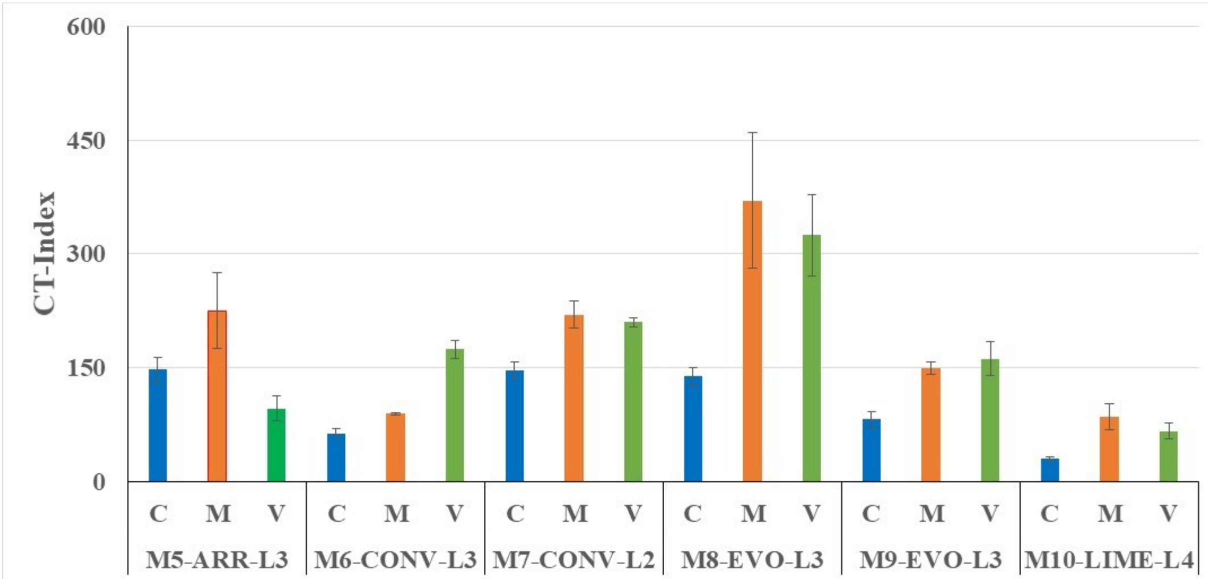


Figure 4.25: CT-index outputs from the IDT test results for the mixtures M5 to M10 (length of the error bar is equal to one standard deviation)

Note: C: Control; M: MIST-2 moisture conditioning at 50ºC and 70psi; V: Vacuum saturation according to AASHTO T 283 (2022).

Table 4.9 shows the TSR values measured at the asphalt production plants (AASHTO T 283, 2014 - Plant vacuum) and the TSR values measured at the OSU Asphalt Materials and Pavements (OSU-AMaP) laboratory. All the TSR values from all conditioning methods are more than 80%, which means they all met the specification requirement (>80%). Most of the plant-measured TSR values are close to the TSR values measured at the OSU-AMaP laboratory.

Table 4.9: TSR Values (%) for Different Conditioning Methods

ID	Plant Vacuum	MIST1	MIST2	Lab Vacuum
M1	-	96.0	93.0	92.6
M2	-	87.8	85.8	82.6
M3	90.0	95.1	87.5	88.7
M4	84.0	96.6	95.2	87.3
M5	-	-	97.7	86.6
M6	85.8	-	90.9	90.3
M7	97.0	-	97.5	97.6
M8	92.0	-	92.7	92.4
M9	99.6	-	98.0	98.9
M10	92.0	-	89.5	95.2

Note: - not available

4.6.2.2 Resilient Modulus

The resilient modulus test determines the elastic modulus (stiffness) of a material based on the recoverable portion of strain during cyclic load application. In NCHRP Report 589, Solaimanian et al. (2007) compared the effectiveness of TSR, HWTD, and ECS/Dynamic Modulus tests in quantifying the moisture resistance of asphalt mixtures. It was concluded that ECS/Dynamic Modulus is the most effective method, while the TSR and HWTD methods have equal effectiveness. It was concluded that the elastic modulus value measured without creating any permanent damage to the asphalt material is more effective in providing the moisture susceptibility of the asphalt mixture when compared to the TSR value. More details about this study are available in Sections 2.3.7 and 2.3.8. In this section, resilient modulus tests with specimens with and without vacuum conditioning were conducted to determine the effectiveness of this parameter for moisture susceptibility quantification. The results of the resilient modulus test for dry (control or C) and vacuum conditioned (V) PMLC samples are shown in Figure 4.26.

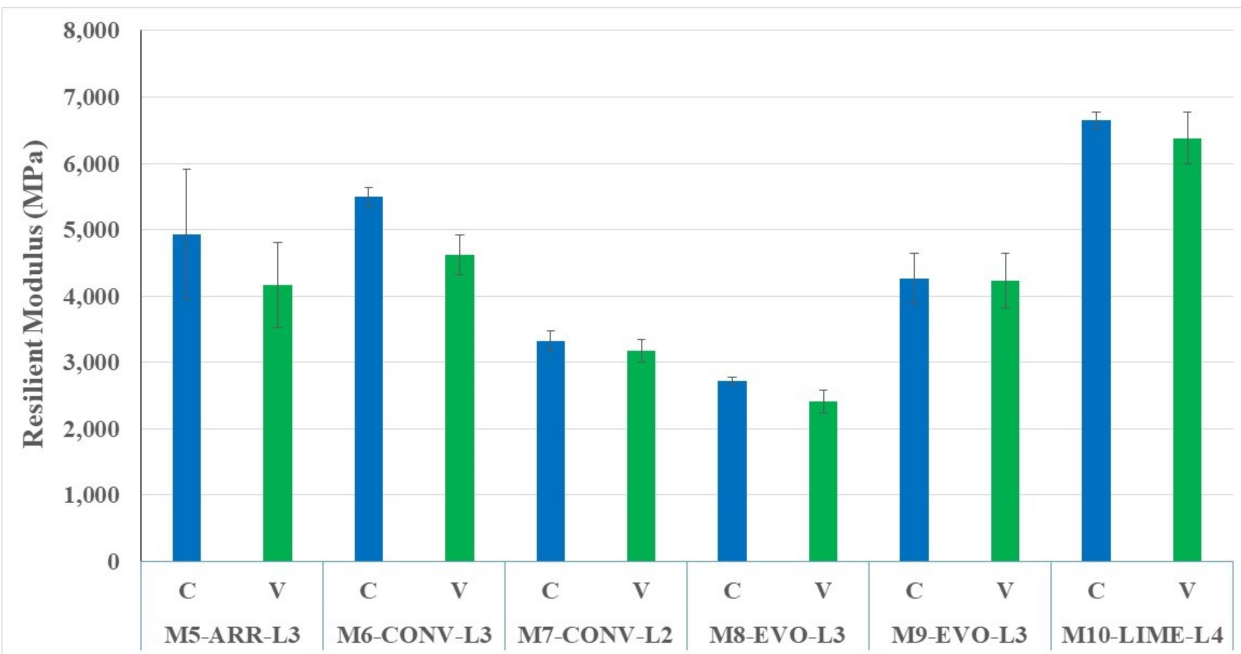


Figure 4.26: Resilient modulus test results for dry (C) and vacuum conditioned (V) plant mixed-laboratory compacted (PMLC) samples

As it can be seen in Figure 4.26, the resilient modulus of the vacuum conditioned samples for all the mixes is lower than the unconditioned samples. The trend observed among the mixes is similar to the strength test results shown in Figure 4.22. The loss of resilient modulus after conditioning is attributed to the loss of adhesion and cohesion in the asphalt specimen. Since TSR also reflects the impact of these two mechanisms in terms of the loss of strength due to moisture conditioning, the correlation between the percentage difference in the resilient modulus for conditioned and unconditioned specimens (called % RM retained) and the laboratory TSRs was investigated. The correlation plot is shown in Figure 4.27. It can be seen from Figure 4.27 that there is an excellent correlation (R^2=0.89) between the percentage resilient modulus retained and the TSR values. The performance rankings based on the two parameters are also similar (although not identical). The TSR rankings (in ascending order-worst to best) for both parameters are given as follows:

%RM retained ranking:

M6 M5 M8 M7 M10 M9

TSR ranking:

M5 M6 M8 M10 M7 M9

These results suggested that resilient modulus testing is not providing any information significantly different from the TSR values. Since resilient modulus testing requires an expensive hydraulic or pneumatic test system, it is not recommended for moisture susceptibility testing in this study.

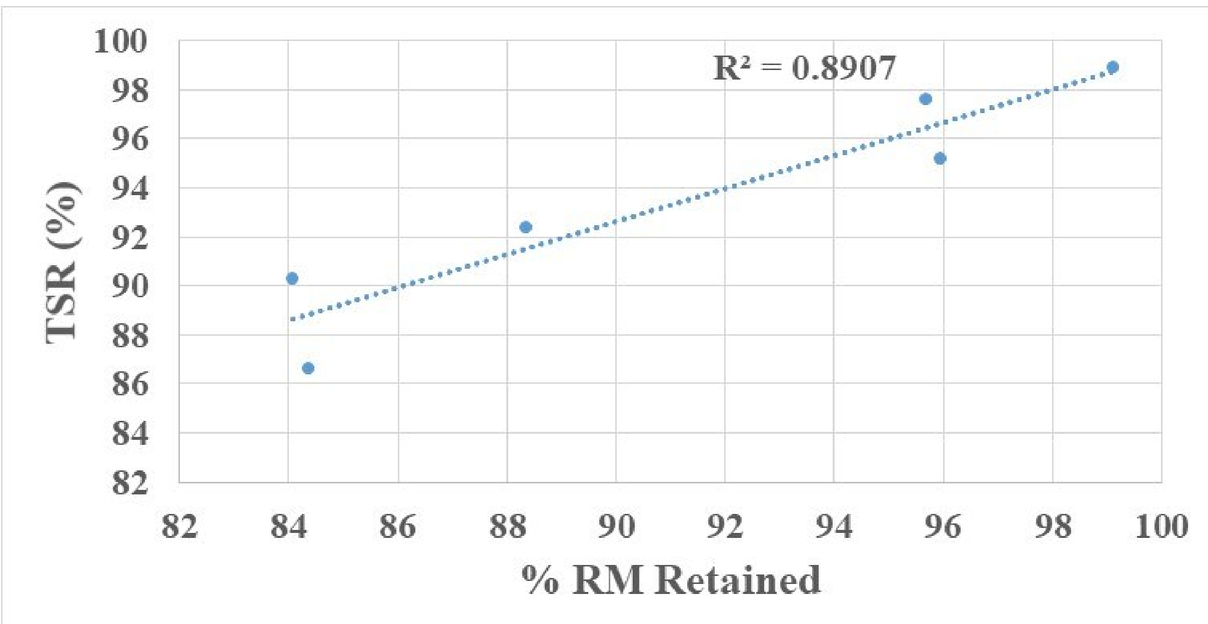


Figure 4.27: Correlation between percentage resilient modulus retained and TSR values

4.6.2.3 HWTT Results

Figure 4.28 shows the results from HWTT performed according to AASHTO T 324 (2019b). It can be observed from the plot that, after 20,000 load applications, samples conditioned with MIST2 and Vacuum had higher accumulated rut depths than the control mixes with no conditioning. The control samples (without any conditioning) for the M9 mix showed similar rutting performance as the samples conditioned in MIST2 or vacuum chamber. This minute change in rut depth due to conditioning points out the low moisture susceptibility of M9. This high moisture resistance is expected to be a result of the

Evotherm additive used in the mixture production. However, a similar result was not observed for M8, which also had an Evotherm additive in it.

Overall, M7 samples showed the highest rut depths among all the mixes with and without conditioning. This is because M7 is a softer mix with less RAP designed for Level 2 (lower volume) traffic. Moreover, M7 does not have any anti-stripping agent such as lime or warm mix additive. M10 showed the lowest rut depth among all the mixes. M10 is a Level 4 lime treated mix with polymer-modified asphalt, and therefore, low rut depth was expected.

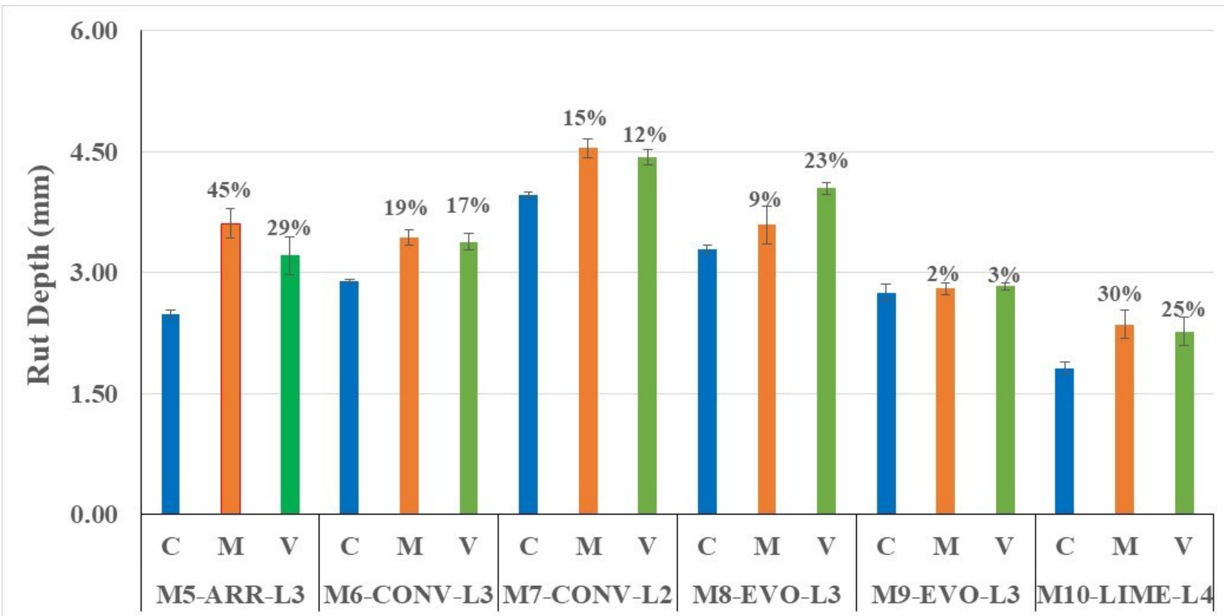
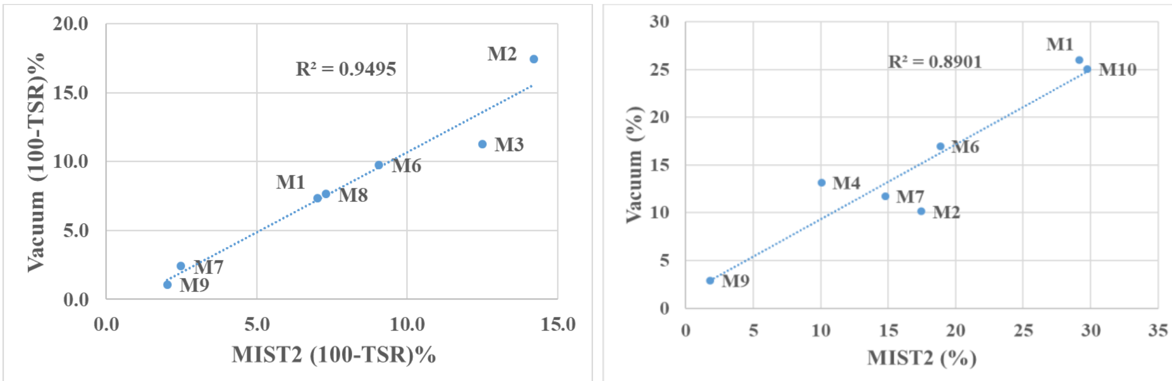


Figure 4.28: HWTT results for different conditioning methods (Phase II)

Note: C: Control; M: MIST-2 moisture conditioning at 50ºC and 70psi; V: Vacuum saturation according to AASHTO T 283 (2022).

The percent difference in rutting numbers at the top of each bar in Figure 4.28 shows the difference in rutting depth between the control samples and the conditioned samples. It can be observed from Figure 4.28 that the percentage increase in rut depths due to conditioning for Mix 8 (Evotherm) and Mix 10 (Lime) are higher than the percentage increase for Mix 6 and Mix 7, which did not have any additives. Thus, HWTT results with the percentage change in rut depth parameter do not identify the impact of additives on moisture susceptibility. The MIST2 conditioned samples of M5 exhibit the highest difference in rutting from the corresponding control samples among all mixes i.e., 45 percent. This can be interpreted as MIST2 conditioned samples for M5 having the lowest moisture resistance compared to the control samples. Similarly, the highest moisture resistance was shown by the MIST2 conditioned samples for M9 (assuming that the parameter reflects the moisture susceptibility). Since Mix 5 and Mix 6 were identical mixtures, the only difference being the ArrMaz antistrip additive in Mix 5, it can be concluded that the “percentage change in rut depth” parameter does not reflect the moisture susceptibility of asphalt mixtures.

Figure 4.29 shows a one-to-one correlation plot of MIST2 and Vacuum conditioning for percent reduction in strength (100-TSR) and percent increase in rut depth. The correlation has been obtained for seven out of ten mixes for both parameters.



(a) **(b)**

Figure 4.29: One-to-one comparison of MIST and Vacuum conditioning based on (a) Percent reduction in strength (100-TSR) (b) Percent increase in rut depth

By comparing the conditioning methods and corresponding rut depths, it can be observed that there is a statistically significant correlation between the results for MIST and vacuum conditioned specimens from both HWTT and TSR. Based on the tests conducted on the samples prepared from a variety of asphalt mixtures and these correlations, it can be concluded that vacuum conditioning is a cheaper and yet very effective conditioning method for evaluating the moisture susceptibility of asphalt mixtures. MIST2 and vacuum conditioned samples showed similar susceptibility towards rutting and moisture. However, MIST2 equipment is costly, whereas vacuum conditioning setup is significantly cheaper, and the process is simpler as well.

4.6.2.3.1 Two-stage Weibull analysis

The results of the two-stage Weibull analysis for Phase II mixes are presented in Figure 4.30 below. For the control cases, Mixes 8, 9, and 10 with additives have lower ratios pointing out better moisture resistance for those mixtures, while the B1/B2 ratios for M6 and M7 (conventional mixes with no additives) are significantly higher. This result points out that B1/B2 ratio and Weibull analysis might identify the impact of additives on moisture susceptibility. However, it should be noted that the same B1/B2 parameter in Phase I was not able to separate the mixtures with additives. To evaluate the effectiveness of the parameter, B1/B2 ratios for all mixtures were compared to the TSR-Vacuum results. Figure 4.31 shows the plots comparing B1/B2 ratios to the TSR-Vacuum results. It can be observed that there is no correlation between the B1/B2 ratios from the Control specimens (meaning no conditioning) and the TSR-Vacuum results. However, the B1/B2 ratios from the HWTT test specimens conditioned with vacuum conditioning are highly correlated with the TSR-Vacuum results. These results point out that submerging and testing HWTT specimens in water does not create internal damage equivalent to Vacuum saturation. For this reason, exposing the specimens to vacuum conditioning resulted in HWTT B1/B2 ratios that are highly correlated with the TSR results. This conclusion also points out the potential issues with implementing HWTT as a moisture susceptibility test without any prior conditioning. A similar conclusion was also derived in Section 4.6.1.2.1 based on the water content (saturation) of tested HWTT cores.

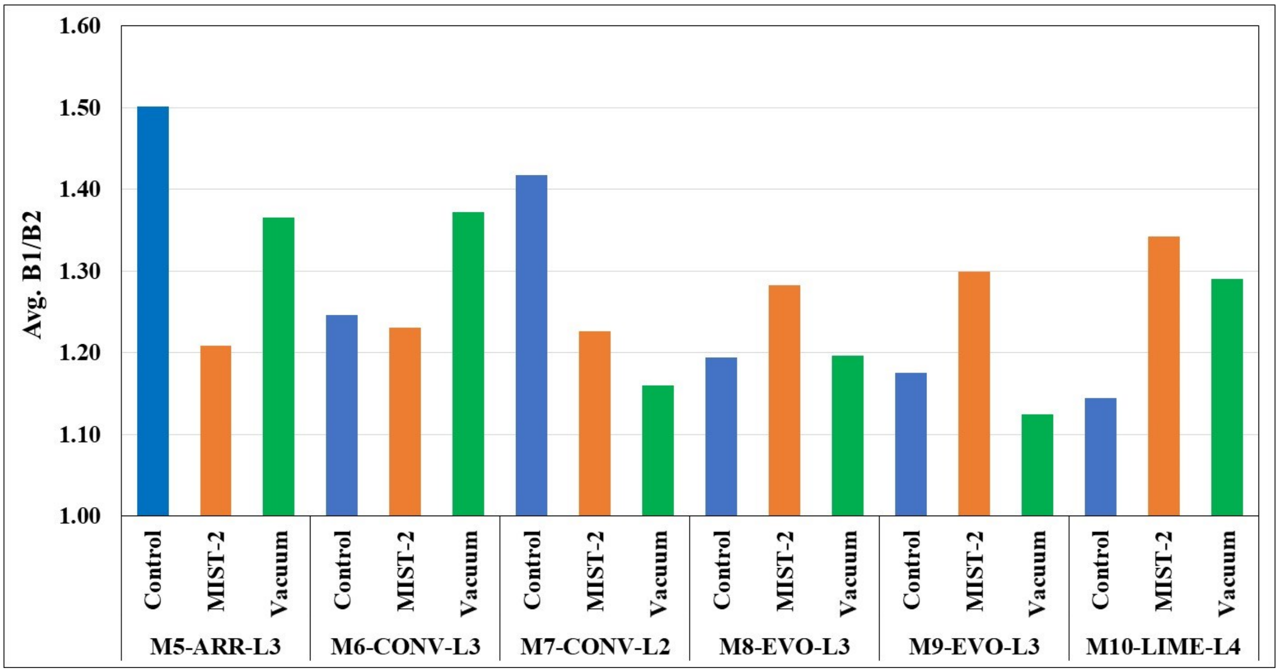
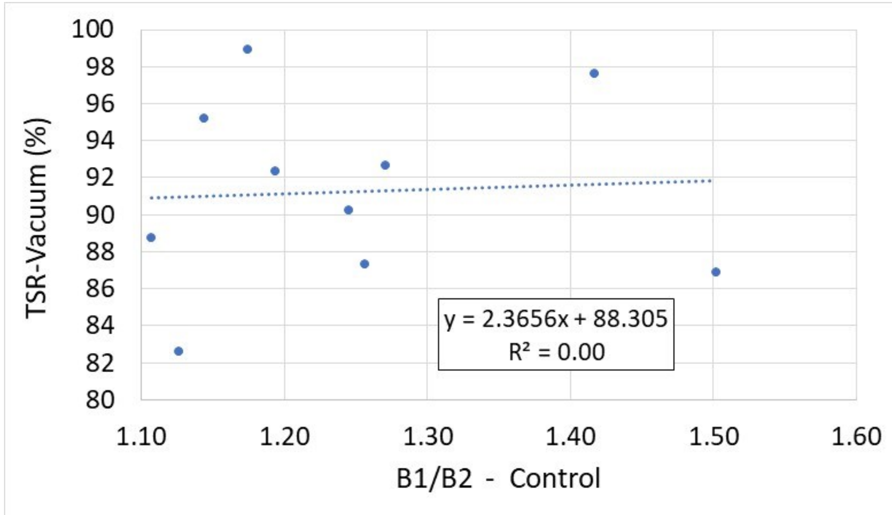
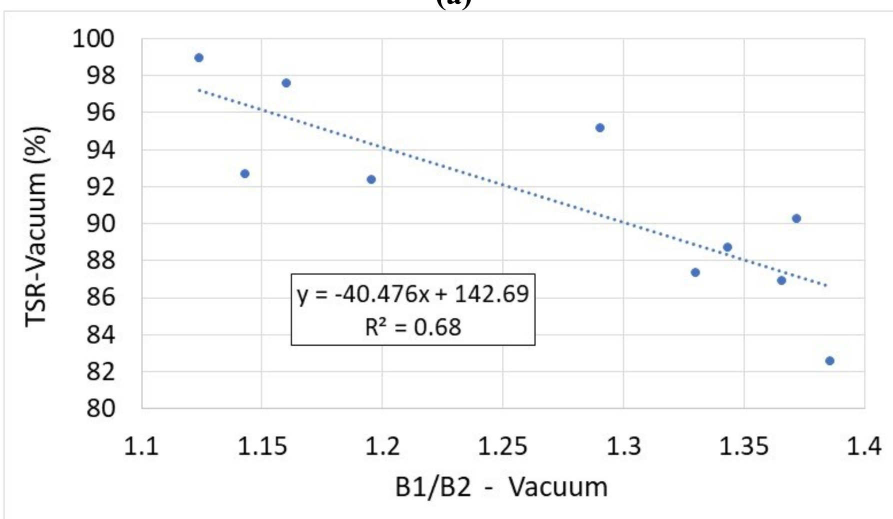


Figure 4.30: B1/B2 Weibull parameter for MIST-2 and Vacuum conditioning methods for mixes 5 to 10



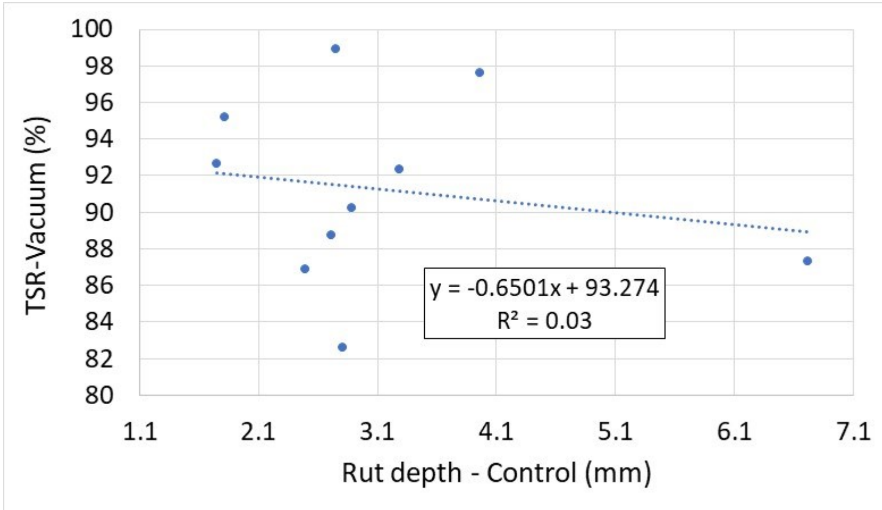
(a)



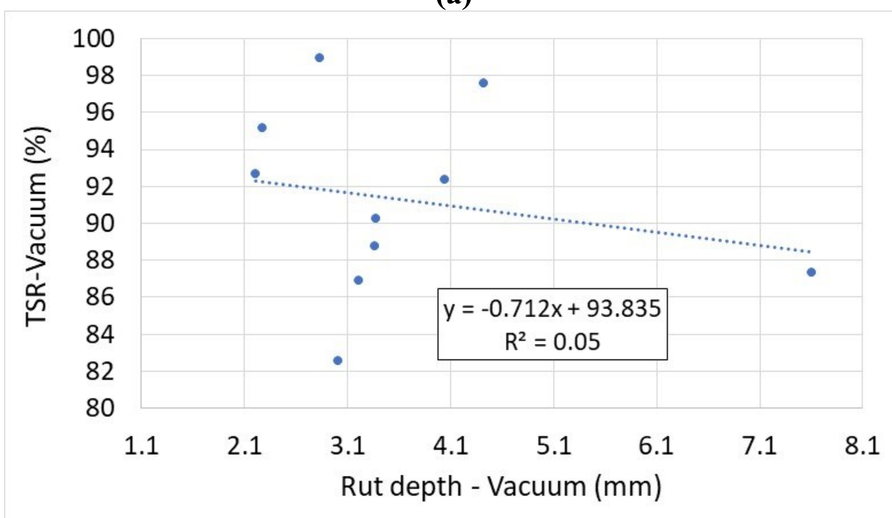
(b)

Figure 4.31: Comparison of B1/B2 Weibull parameter with TSR-Vacuum (AASHTO T 283, 2014) results (a) B1/B2-Control versus TSR-Vacuum (b) B1/B2-Vacuum versus TSR-Vacuum

To determine the effectiveness of "Rut depth" parameter for moisture susceptibility evaluation, the correlations between "Rut depth-Control" and "TSR-Vacuum" and "Rut depth-Vacuum" and "TSR-Vacuum" were also checked (See Figure 4.32). It can be observed that the "Rut depth" parameter is not correlated with the TSR values. This result points out that B1/B2 ratio might be a better parameter to quantify the moisture susceptibility of different asphalt mixture types when compared to the "Rut depth" parameter. This statistically significant correlation between "B1/B2-Vacuum" and "TSR-Vacuum" proves that the Vacuum saturation method can provide consistent results no matter what the test method is.



(a)



(b)

Figure 4.32: Comparison of rut depth parameter with TSR-Vacuum (AASHTO T 283, 2014) results (a) Rut depth-Control versus TSR-Vacuum (b) Rut depth-Vacuum versus TSR-Vacuum

4.6.3 Phase III

In this phase of the study, the boiling test method was performed on the loose mixes by following the procedures described in Sections 2.3.1 and 4.5.4. The L* (or color index) values of unboiled loose asphalt mixtures and boiled asphalt mixtures were recorded using a Colorimeter (see Figure 4.8). Colorimeter measurements were also conducted on the broken asphalt cores from the TSR tests (vacuum conditioned and control specimens), results of which are presented later in this section.

The findings from the boiling tests are shown in Figure 4.33. It should be noted that error bars (brackets) are not provided on every bar to show the variability since the colorimeter readings were expected to be highly variable due to the significant color difference between the stripped and unstripped locations on the aggregates. However, the average of the several measured values is expected to represent the overall color of the material before and after conditioning. Table 4.10 shows the average percentage stripping (L*RB) values of the mixes calculated by using Equation 4-7 with the TSR-vacuum results. Results show that the boiled loose asphalt mixtures exhibited a greater average L* value in all cases. However, the percentage change in L* values is not pointing out the impact of lime and chemical additives on the moisture susceptibility of the asphalt mixtures (in other words, not separating the modified mixes from the unmodified ones). According to the L*RB values in Table 4.10, Mix 8 should be exhibiting the highest stripping. Moreover, the mixes without any additives (Mix2, Mix 6, and Mix 7) have significantly lower percentage stripping values. Thus, the Boil test conducted on the loose mixtures does not seem to be a reliable test as it is not able to draw a clear distinction between mixes with and without additives. As discussed in Section 2.3.1, the boiling test method is not effectively simulating the stripping happening in the field under high pore pressures created by heavy truck loads. It is also important to note that no visible loss of binder coating was observed after boiling for any of the mixes. The observed differences in L* values were captured by using a sophisticated Colorimeter. It should also be noted that the L*RB values are not correlated with the TSR-Vacuum results (See Table 4.10).

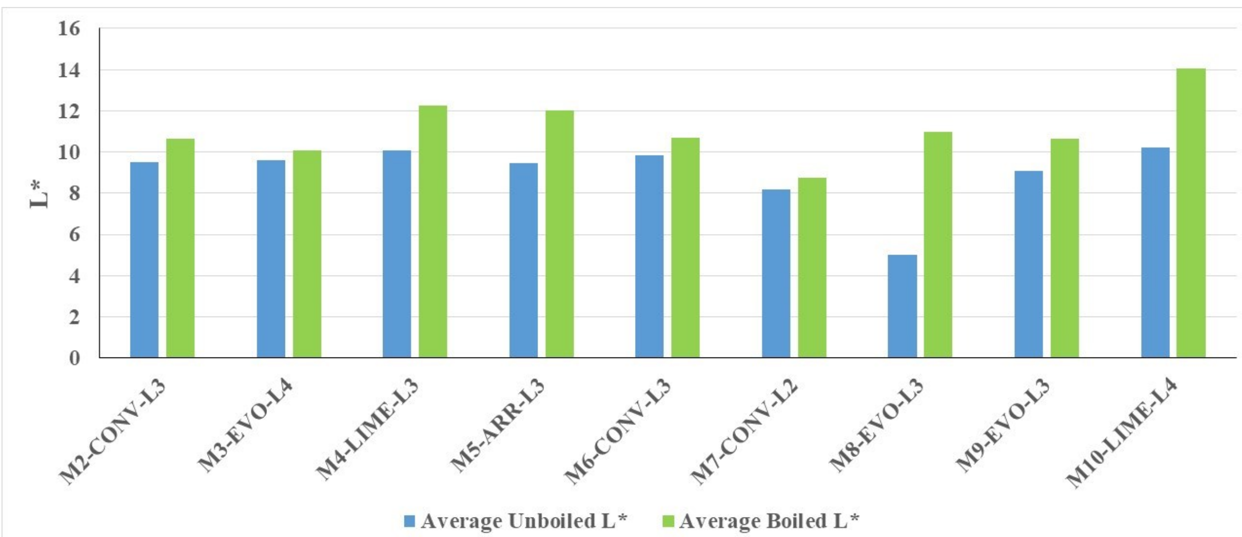


Figure 4.33: Boil test result for the loose mixes

Table 4.10: Average Percentage Stripping (L*RB) Values from The Boiling Test

Mix	Average Unboiled L*	Average Boiled L*	L*RB	TSR Lab (Vacuum)
M2-CONV-L3	9.48	10.66	12.39	82.6
M3-EVO-L4	9.59	10.07	5.02	88.7
M4-LIME-L3	10.07	12.25	21.66	87.3
M5-ARR-L3	9.46	12.01	26.93	86.6
M6-CONV-L3	9.83	10.71	8.90	90.3
M7-CONV-L2	8.20	8.77	6.90	97.6
M8-EVO-L3	4.98	11.00	120.64	92.4
M9-EVO-L3	9.10	10.65	17.12	98.9
M10-LIME-L4	10.21	14.03	37.50	95.2

The colorimeter readings were also obtained on the fractured IDT samples. The two broken halves of the tested IDT samples were treated as two replicates, and several color readings were collected from the broken TSR specimens for the mixes 5 through 10. Readings were taken on both the unconditioned and the conditioned samples. The results are shown in Figure 4.34.

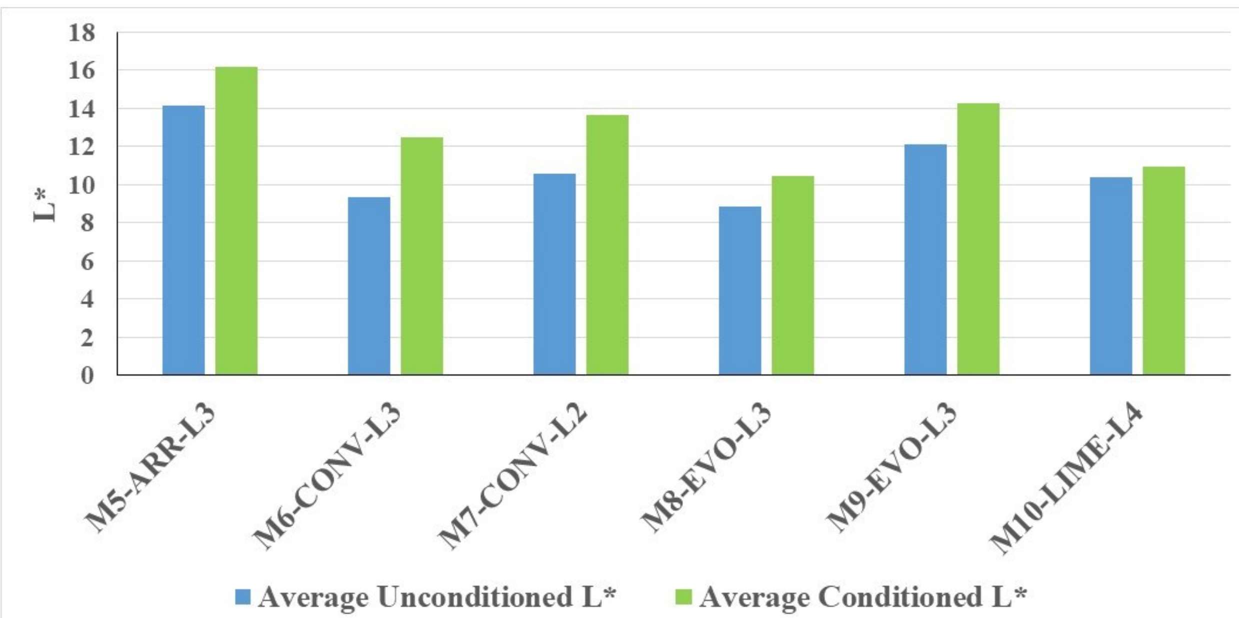


Figure 4.34: Colorimeter readings on tested IDT samples

It can be seen from Figure 4.34 that the L* values of the moisture-conditioned samples were greater than the unconditioned samples for all the mixes. Unlike the loose asphalt mixtures, stripping of binder from the aggregate surface was also visible to the naked eye (see Figure 4.10). This stripping is expected to be correlated with adhesion-related moisture failures, which is expected to be the major reason for in-situ moisture-related failures (Hicks 1991).

The average percentage stripping was also calculated for all the broken samples based on the observed L* values, and the results are shown in Figure 4.35.

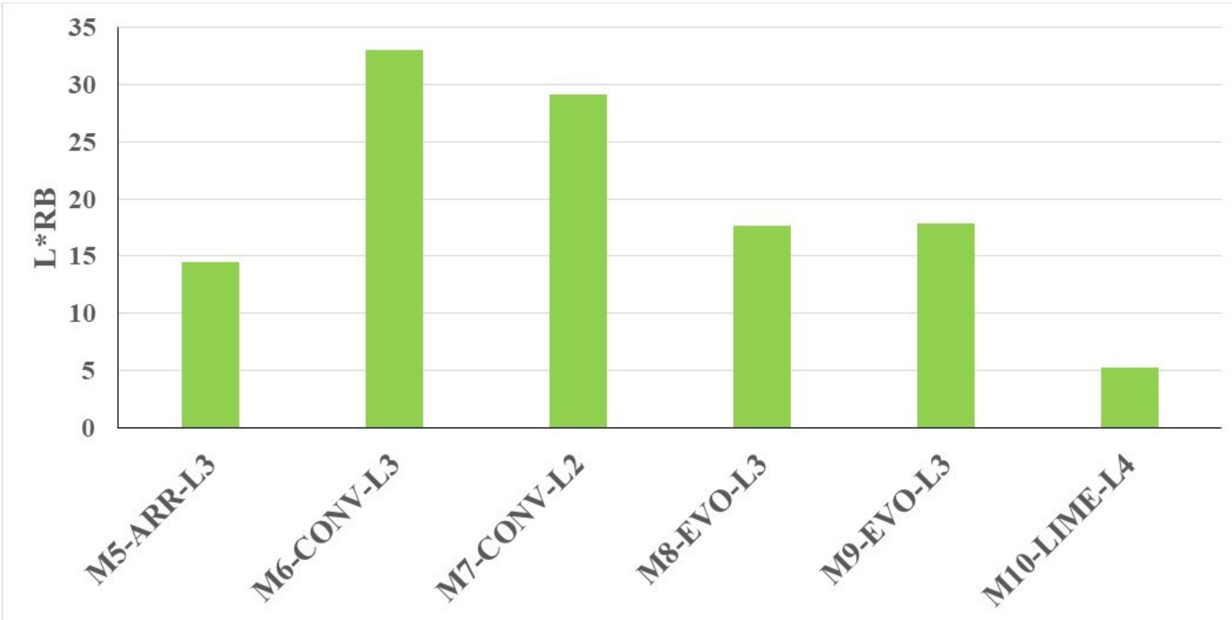


Figure 4.35: Average L*RB results (expected to be correlated with adhesion-related stripping) from the tested IDT samples of different mix

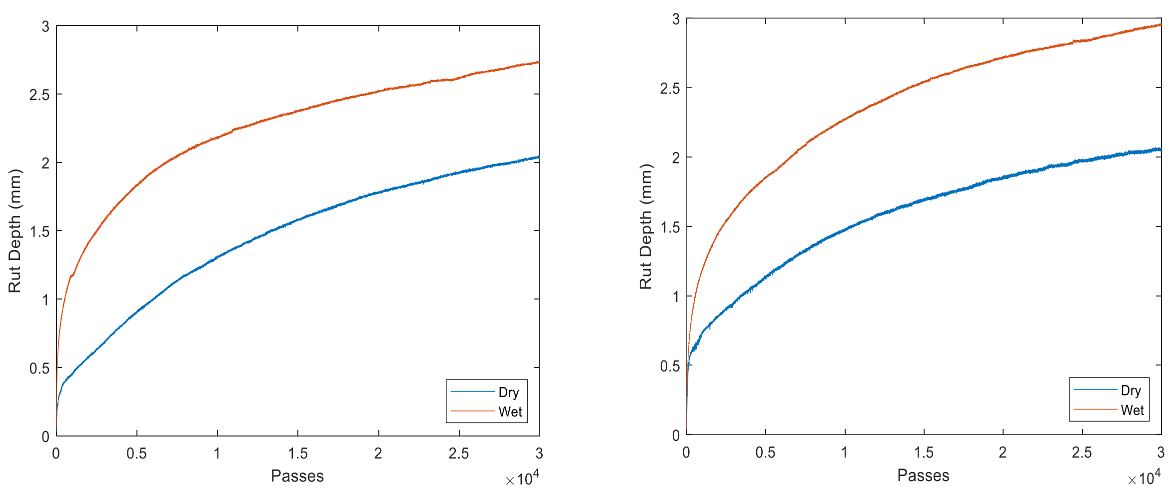
From Figure 4.35, it can be seen that the mixes without any additives showed the highest stripping among all the mixes. Mix 10 had the lowest average percent stripping, and mixes with Evotherm (Mix 8 and Mix 9) showed comparable percent stripping values. According to the results, Mix 5 with liquid antistrip performed better than the mixes with Evotherm, while the use of lime in the asphalt mixtures resulted in the lowest level of adhesion loss. M5 and M6 were produced at the same plant and had the exact same ingredients except the additional antistripping (ARR) additive used in M5. Due to the presence of liquid antistrip in M5, it was expected to have a higher TSR value (meaning higher moisture resistance). However, the TSR value for M5 was lower than M6. It can be observed that the L*RB parameter calculated from the colorimeter readings was able to show the positive impact of the ArrMaz additive on stripping resistance. Thus, the colorimeter readings on the fractured IDT samples were able to separate the mixes with and without additives. However, the results obtained from the compacted and conditioned samples were not found to be correlated with the TSR values of the same samples. As described in Section 4.6.2.1, TSR results are highly affected by the mixture variables (binder content, RAP content, D-to-B ratio, etc.). In addition, TSR values are expected to be affected by both the cohesion and adhesion-related damage, while the L*RB values from the broken cores are expected to represent only the adhesion-related failures (separation of binder from the aggregates). However, since most of the moisture-related failures in Oregon result from adhesion-related failures, L*RB values are expected to complement the TSR results and provide a quick and easy method to determine adhesion-related stripping in fractured samples. The repeatability and reliability of using the L*RB parameter for moisture susceptibility evaluation should be further explored in a future research study with cores extracted from pavements with known historical performance against moisture.

4.6.4 Phase IV

4.6.4.1 Dry and Wet-Vacuum HWTT

It is hypothesized that standard HWTD tests conducted in water simulate both moisture and rutting-related deformation. To isolate the moisture-related failure from the moisture and rutting-related failure, Lu (2005) suggested conducting HWTD tests in both dry and wet conditions. Since dry tests are expected to only simulate rutting resistance-related failures, subtracting the area under the average dry test rutting curves from the area under the wet test rutting curves is expected to provide a parameter correlated with the moisture susceptibility of the asphalt mixture (higher numbers being more susceptible).

Figure 4.36 illustrates the dry and wet rutting curves for Mix 2 and Mix 3. These curves are obtained by averaging the rutting curves from all replicate test results. It can be seen from Figure 4.36(a) and Figure 4.36(b) that the maximum rut depths of the two mixes are higher in wet conditions than in the dry tests (as expected).



(a) **(b)**

Figure 4.36: Average rutting accumulation curve from dry and wet-Vacuum HWTT (a) MIX 2-CONV-L3 (b) MIX 3-EVO-L4

The major difference between the dry and wet rutting curves is the rate of deformation accumulation. In wet conditions, the creep accumulation is faster, as can be seen from the initial and secondary stages of the rutting curves in Figure 4.36. The faster accumulation of deformation for the wet tests can be the result of moisture damage in wet conditions. This impact of moisture can thus be parametrized by the difference in the area (ΔA_{RD}) between the wet and dry rutting curves. Figure 4.37 shows the difference in area between the wet and dry rutting curves for all the six mixes. Higher ΔA_{RD} signifies higher moisture damage potential.

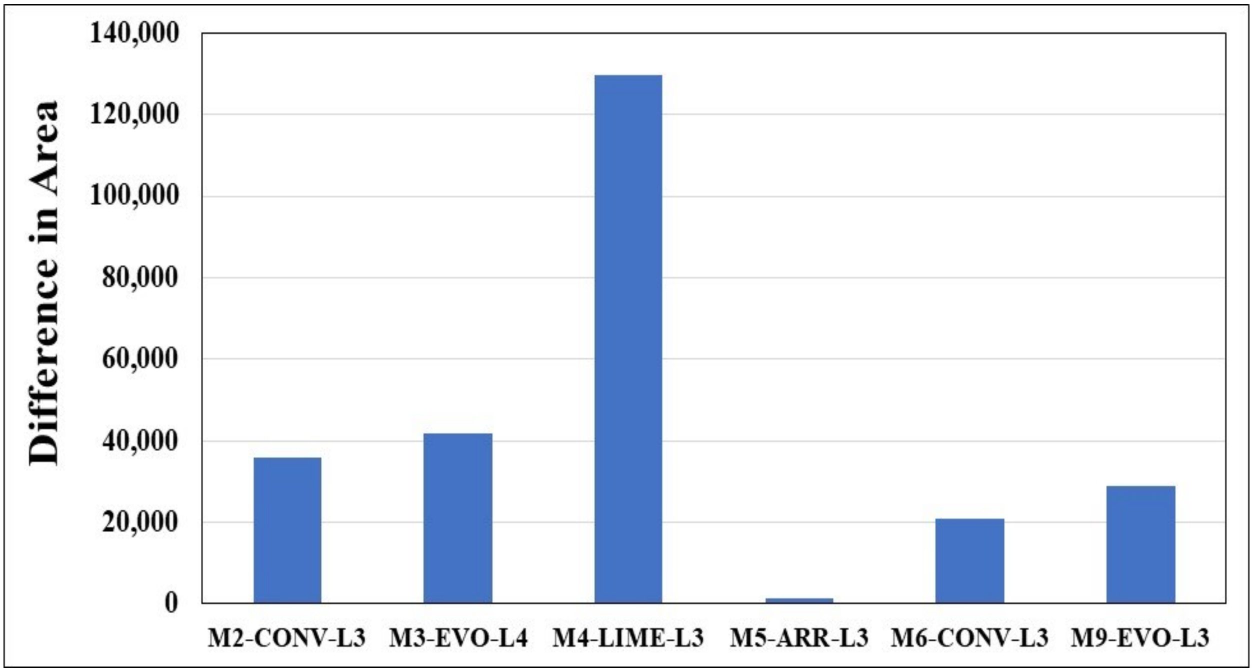


Figure 4.37: Difference between the area under the dry (no prior vacuum conditioning) and wet rutting curves (vacuum conditioned before testing)

It can be seen from Figure 4.37 that ΔA_{RD} for Mix 4 was the highest among all mixes, although Mix 4 had lime in it. M3 with the Evotherm additive also had the second-worst moisture resistance out of all 6 mixes evaluated in this part of the study. Since Mix 3 had the Evotherm additive, it was expected to perform better than the conventional mix (Mix 2). However, no significant difference between Mix 2 and Mix 3 was observed (although Mix 3 had a slightly higher ΔA_{RD}). Although these results might be pointing out the issues with using the wet and dry HWTT results for moisture susceptibility evaluation, some of the results provided more reasonable conclusions. For instance, ΔA_{RD} for Mix 9 (the mix with Evotherm additive with the highest TSR value) was significantly lower than Mix 2, Mix 3, and Mix 4. This agrees with the TSR value of Mix 9 (98.9%), which is much higher than Mix 2 (82.6%) and Mix 3 (88.7%). However, the overall trend in rankings of the mixes based on ΔA_{RD} is not similar to that based on TSR.

It can also be observed from Figure 4.37 that Mix 5 with the ArrMaz additive had the lowest ΔA_{RD} when compared to all other 5 mixes. Since Mix 5 and Mix 6 had identical mix designs except for the ArrMaz additive in Mix 5, ΔA_{RD} parameter can be expected to be capturing the impact of this additive on moisture susceptibility.

As discussed earlier, ΔA_{RD} values can be significantly affected by the maximum rut depth shown by the mixes. For a highly rut-resistant mix, the difference in the area for the wet and dry results can be small due to the high rut resistance. To eliminate this bias, ΔA_{RD} was normalized by dividing the parameter by the area under the dry test curve and multiplying the output by 100. The final value is expected to be correlated with the moisture susceptibility of the asphalt mixture. Results are shown in Figure 4.38.

Although this parameter is expected to eliminate the rut resistance related bias, it did not provide any conclusions different from the ones listed above for the ΔA_{RD} parameter shown in Figure 4.37 .

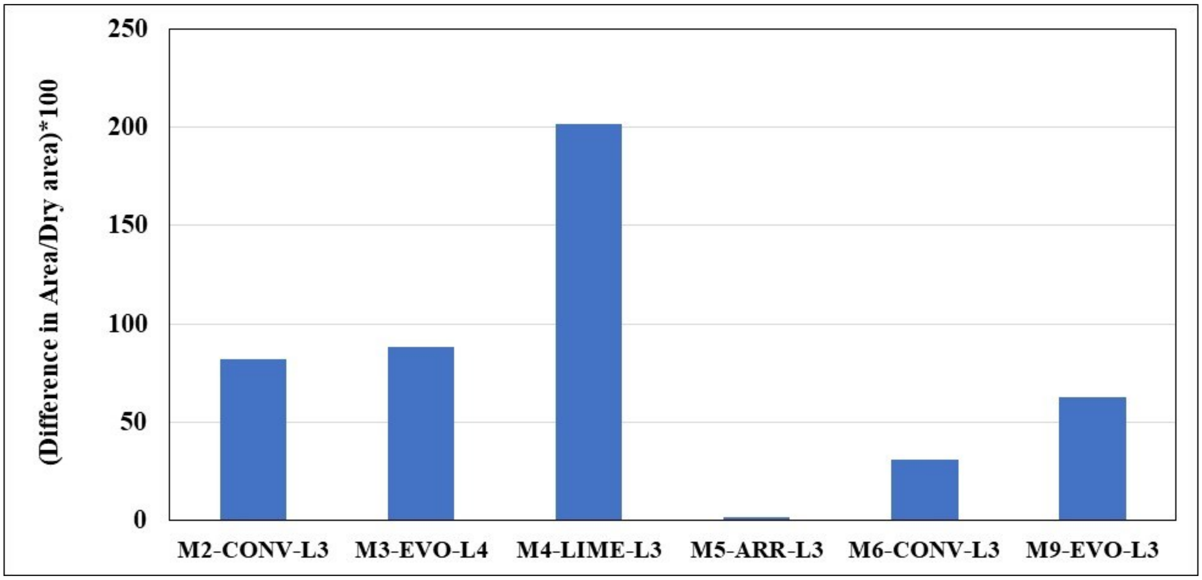


Figure 4.38: Difference between the area under the dry (no prior vacuum conditioning) and wet rutting curves (vacuum conditioned before testing) divided by the dry area

Based on the results, it is not possible to confidently conclude that the ΔA_{RD} parameter is capturing the impact of additives on moisture susceptibility. However, as also previously discussed, the use of additives may not always improve the moisture resistance due to problems with the mix design parameters or variability during production. For this reason, in a future study, these results should be compared with the in-situ long-term moisture susceptibility of pavements constructed with these 6 mixes.

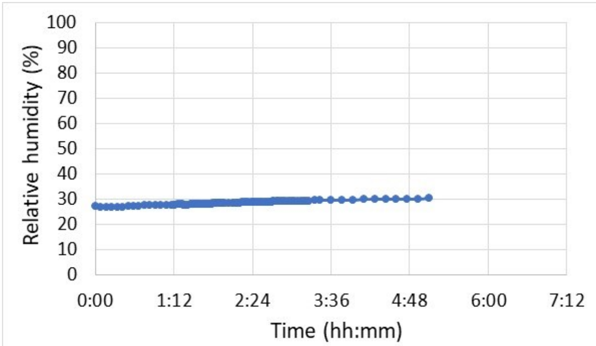
4.6.4.2 In-Situ Infiltration

Many studies have correlated poor subsurface drainage to accelerated asphalt pavement degradation (Hicks, 1991; Scholz & Rajendran, 2009; Brown, 2010). To mitigate excessive moisture damage, adequate drainage of the pavement surface and surrounding area is necessary. Effective drainage is achieved by implementing multiple systems working in tandem to remove excess water and prevent pooling at or around the pavement surface (See Sections 2.2.7 and 2.2.8 for more literature on this issue). Pavement density also plays a vital role in keeping the functionality of the drainage system by removing the water on the pavement surface without having any significant infiltration into the layers. Failure of the water drainage process and getting excessive water into the asphalt microstructure and the bond layer between the pavement layers can result in early moisture-related failures. For this reason, monitoring the moisture infiltration into the asphalt layer is critical. In this study, a special sensor system was used to determine the effectiveness of these sensors for moisture infiltration monitoring. The detailed procedure followed for moisture infiltration monitoring using the proposed sensors is given in Section 4.5.6.

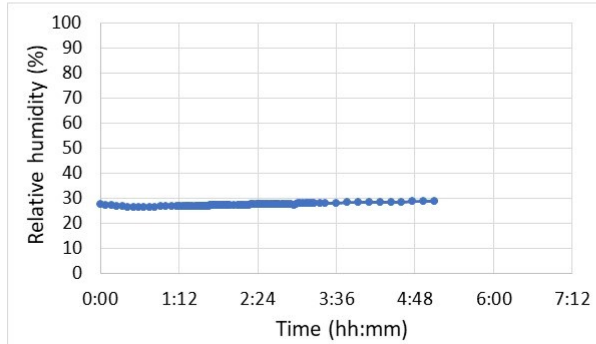
The results from the infiltration tests conducted by using field moisture sensors are presented in Table 4.11. The progression of moisture infiltration over time is presented in Figure 4.39. All tests were carried out for a duration of six hours. The control sensor measured the ambient relative humidity based on the change in electrical impedance. As expected, the sample with 15% air void content showed a considerable rise in relative humidity within the asphalt block as captured by the two replicate moisture sensors. Water from the rainfall simulator flowing over the block in horizontal as well as vertical directions infiltrated into the interior of the block, changing the relative humidity within the sample. Sensor 1 recorded the lowest difference (35.1%) in the relative humidity in the 15 percent air void block. The second sensor recorded a difference of 69.4% relative humidity. It might be possible that one side of the sensors received more water due to the variability in the porosity within the asphalt microstructure (spatial density variability). In 5, 7, and 9 percent air void blocks, both sensors detected only minute changes in relative humidity. This finding is expected to be a result of the high density of these asphalt blocks. Although there is still air space in the blocks, the level of air void content is not enough to connect the air void channels, which is necessary to increase the permeability. However, it should be noted that data was collected only for a 6-hour period. For longer simulated rainfall events, it might be possible to observe water infiltration for the asphalt block with 9% air void content.

Table 4.11: Moisture Infiltration Test Results Recorded by Moisture Sensors

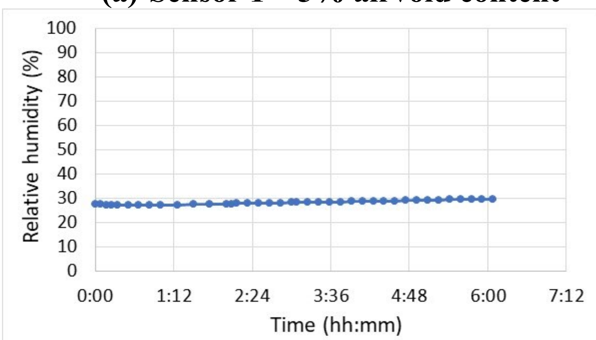
% Air voids in the sample	**The difference in Relative Humidity (RH) recorded at the beginning and the end of the test**		
	Sensor 1	**Sensor 2**	**Control Sensor**
5	2.4	3.3	4.9
7	2.2	0.6	0.9
9	3.2	1.2	1.9
15	35.1	69.4	3.6



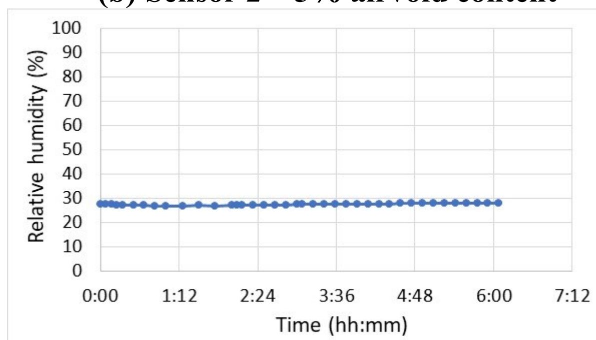
(a) Sensor 1 – 5% airvoid content



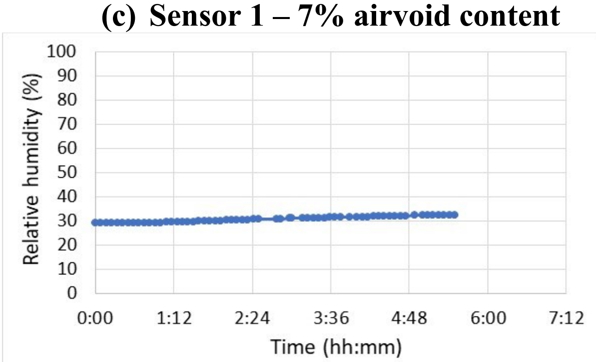
(b) Sensor 2 – 5% airvoid content



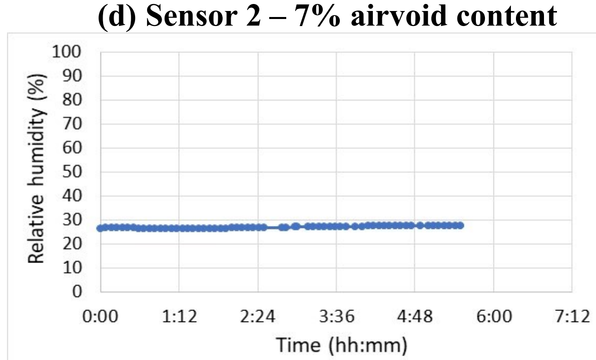
(c) Sensor 1 – 7% airvoid content



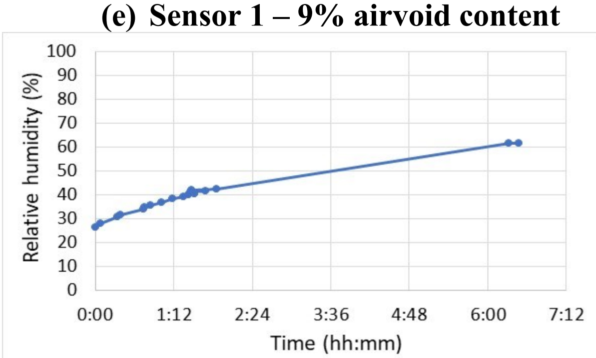
(d) Sensor 2 – 7% airvoid content



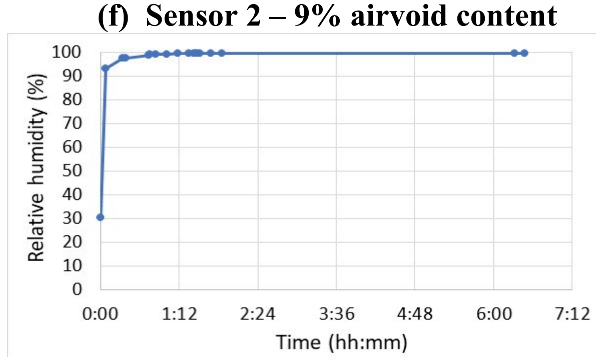
(e) Sensor 1 – 9% airvoid content



(f) Sensor 2 – 9% airvoid content



(g) Sensor 1 – 15% airvoid content



(h) Sensor 2 – 15% airvoid content

Figure 4.39: Moisture sensor data – time versus relative humidity

The merit of this test lies in its ease of use and operation. Although the test is destructive in nature, it does not require the removal of full-depth cores. It can be readily applied in the field. The recordings are displayed in real-time on the cellphone screen via Bluetooth connection and a mobile application. If the holes are drilled properly, and the caps of the sensors are sealed securely, this test has a great potential to be used as an alternative to the traditional permeability tests and destructive forensic investigations for moisture damage monitoring. However, more comprehensive testing and field sensor installation and monitoring are required before the implementation of this sensor system for in-situ moisture measurements.

Only three sensors were purchased for a preliminary investigation in this study ($400 price), and the free cellphone app was used for collecting data. The cellphone app is not capable of logging data automatically. For this reason, data was collected manually by checking the app every 2 to 5 minutes and recording the values (for this reason, the data collection duration was limited to 6 hours). For continuous and automated data collection for longer periods, the version of the system with the data acquisition device needs to be purchased (the price for the complete system with five sensors and the data acquisition device is $1,800). However, for field implementation for monitoring the infiltration of water into the critical locations in the pavement structure, the cellphone app use is expected to be the reasonable method, while the data acquisition device can be useful for longer moisture infiltration monitoring in research studies. The sensors can be installed on a highway section at various locations. At different times of the year (before, after, and during the rainy seasons), the moisture level can be measured by wirelessly connecting to the sensors from the side of the road (without needing a traffic closure). In this way, without conducting destructive forensic investigations (coring or trenching), moisture-related failures at the structural level can be easily explained.

4.6.4.3 Electrical Resistivity

Results from electrical resistivity testing are shown in this section. The procedure followed for electrical resistivity measurements is described in Section 4.5.5. In the trial phase, the test was first carried out by saturating the cylindrical samples in water, but the resistivity meter recorded unrealistically high phase angles. To resolve this issue, a standard pore solution was used according to AASHTO TP 119 (2015). The resistivity of the standard pore solution was determined to be 0.127 Ωm. The shape factor of the cylindrical core used in this test was 0.1767 m (for 100 mm tall and 150 mm diameter core). Two thin sponges were used at each end of the core for better contact with the electrodes. The sponges were saturated with the pore solution before placing them on the top and bottom electrodes. The phase angle (Φ) was found to be zero at 1 kHz frequency which implied that the readings recorded by the resistivity meter could be directly used as resistance value instead of impedance (Giatec 2022). The formation factor was calculated corresponding to the recorded resistance values. Table 4.12 summarizes the measured electrical resistances and formation factors for all the tested samples.

Table 4.12: Electrical Resistivity Test Results

Mix ID	Electrical Resistance R (kΩ)	Bulk electric resistivity ρ (Ωm) (k = 0.1767 m)	Pore solution resistivity ρ_s (Ωm)	Formation Factor F (ρ/ρ_s)
Mix 2	1.30	229.7	0.127	1,808
Mix 8	2.75	485.9		3,825
Mix 9	2.81	496.5		3,909
Mix 10	1.05	185.5		1,460

Table 4.12 indicates that Mix 10 showed the lowest electrical resistivity among all the mixes, and Mix 9 showed the highest electrical resistivity. Although all the samples were prepared by compacting the mixes to target 7±0.5% air voids, the difference in electrical resistance of all four mixes is significant. Thus, the microstructure and porosity of these samples can be expected to be different. It can be observed from Figure 4.40 that measured electrical resistivity values are positively correlated with the VFA values measured at the plant (meaning VFA is negatively correlated with porosity). This result suggested that the electrical resistivity measurements can capture the reduction in porosity as a result of the higher VFA. This result shows that electrical resistivity measurements for asphalt mixture porosity evaluation can be a promising method. However, additional research with several other mixture types and density levels must be conducted before adapting this test method for porosity and moisture susceptibility evaluation.

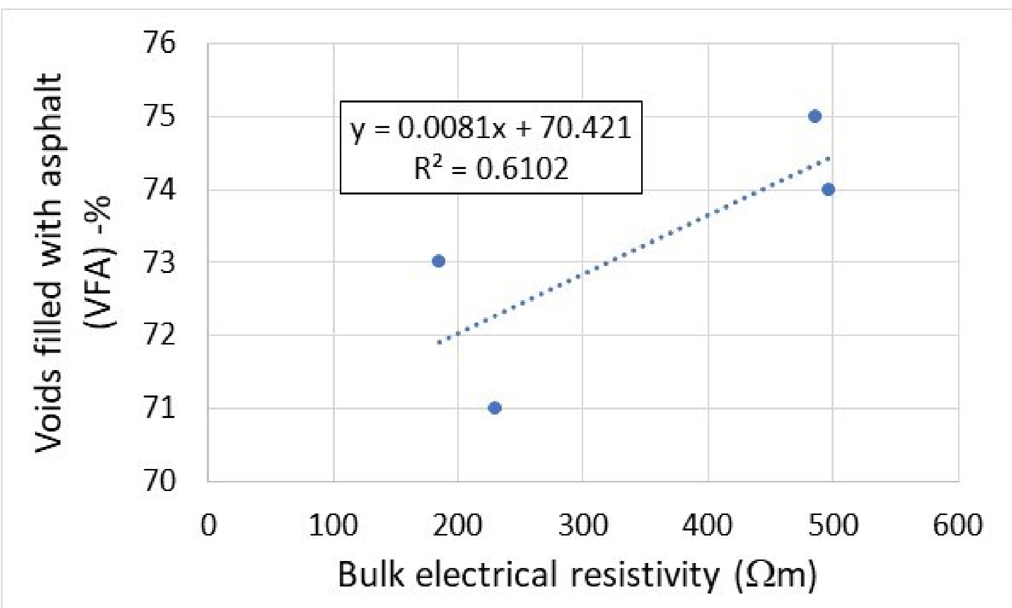


Figure 4.40: Correlation between bulk electrical resistivity and VFA

Some challenges faced during the test were the desorption of pore solution from the asphalt core, the use of sponges, and the varying amount of pressure applied while tightening the bolts of the jig. It is possible that the variations observed in the electrical resistances in the samples were due to any of these reasons. To overcome the issue faced by desorption of pore solution, an effort was made to record the measurement at the end of five seconds after removing the sample from the solution chamber. This ensured consistency in the process.

The successful measurement of the electrical resistivity of the asphalt cores is promising and can provide a major breakthrough in determining the pore interconnectivity of asphalt materials. It should be noted that asphalt concrete materials with the same density level may have different pore connectivity and overall permeability. Formation factor has been related to the properties and durability of concrete, and a careful investigation of similar relationships in asphalt can lead to important findings. In future research, a greater number of replicates and samples with different air void contents and aggregate gradations should be used to determine the effectiveness and accuracy of the test. Results should also be checked towards more accurate (ground truth) methods such as X-Ray CT image data to validate the effectiveness and reliability of the measured parameters.

5.0 CONCLUSIONS, RECOMMENDATIONS, AND FUTURE RESEARCH

Moisture damage in asphalt mixtures can cause early cracking and rutting failures due to the internal damage accumulated by the high internal pore pressures created at the aggregate-binder interface and/or within the binder phase by heavy traffic loads. Due to the high precipitation levels and frequent rain events, distresses originating from moisture damage are commonly observed on roadways in Oregon. ODOT has been mostly using hydrated lime to combat distresses related to moisture damage at the mixture level, while the effectiveness of new chemical anti-strips and warm-mix technologies has also started to be investigated. However, a reliable moisture conditioning method and moisture susceptibility test need to be developed and implemented for Oregon to determine the possible long-term impact of several new additive technologies on pavement longevity. Roadway geometry, asphalt layer density, construction of proper superelevation on the roadway for effective water removal, and functioning drainage facilities can be considered to be the other important factors that control moisture-related failures on roadways. In this study, the most reliable and promising asphalt mixture conditioning and testing methods to evaluate moisture susceptibility are determined. Developed tools and test procedures are expected to help ODOT identify the benefits of recent additive technologies that are being developed to combat moisture damage of asphalt mixtures.

5.1 CONCLUSIONS

The major conclusions drawn from the results of this study are as follows:

Literature review

A comprehensive literature review was conducted in this study and presented in Section 2.0. A summary of the literature review is also given in Section 2.4. The major findings from the literature review are given here:

1. The mechanisms controlling the loss of adhesion and cohesion in the asphalt mixture work in tandem to strip binder from aggregate and damage the binder phase in the asphalt concrete mixture, thereby accelerating pavement degradation (Hicks 1991), while the loss of adhesion was generally accepted to be the major factor controlling moisture susceptibility.

2. Although the loss of adhesion and cohesion are the two major mechanisms causing moisture damage in asphalt mixtures, the delamination caused by water infiltrating through the pavement and damaging the bond between the two pavement layers is also a common mechanism for moisture-related pavement failure (Khosla et al. 1999; Scholz and Rajendran 2009*)*. For this reason, bonding between the pavement layers, density and permeability of the asphalt layers, roadway geometry, and the presence and functionality of drainage facilities along the roadway are also other factors

controlling the moisture susceptibility of asphalt surfaced pavements. It is possible to have a roadway constructed with the least moisture susceptible asphalt mixture and still end up with severe stripping issues due to design and construction mistakes. For these reasons, it is not possible to eliminate moisture damage by just improving asphalt mixtures' moisture resistance.

3. Solaimanian et al. (2003) indicated that ECS conditioning effectively simulates field conditions by pushing the water into the asphalt microstructure and simulating high pore pressures created by truck loads. However, it was also concluded that the correlation between the field moisture susceptibility and the test results from ECS conditioned samples is not higher than the correlation between field performance and the test results for the samples tested with AASHTO T 283 (2014). In addition, the cost of the system is significantly higher than the AASHTO T 283 (2014) setup. AASHTO T 283 (2014)also has a more practical conditioning process than the ECS.

4. AASHTO T 324 (2019b) makes it difficult to identify whether the damage is caused by rutting, moisture, or both. Extensive testing with the HWTD conducted in Europe and the U.S. suggested that the test should only be used for rutting evaluation, as it has a minimal indication of moisture susceptibility (European Committee for Standardization (ECS), 2020 - ''Small size device", Procedure B conducted in the air; Tsai et al. 2016). Lu (2005) also concluded that the total measured surface deformation from the HWTD is a combination of both moisture and rutting-related permanent deformation. Lu (2005) recommended conducting both dry and wet HWTD tests to be able to separate the stripping-related deformation from the rutting-related deformation to quantify the moisture susceptibility of the asphalt mixture. According to the literature review conducted in this research project, HWTD may not provide reliable moisture susceptibility estimates in many cases since test results are controlled mainly by the rutting resistance of the asphalt mixture (See section 2.3.8).

5. Several research studies were conducted in the literature to determine the effectiveness of different testing and conditioning methods for moisture susceptibility evaluation. The majority of these research studies focused on correlating test results with the reported field moisture resistance of asphalt mixtures to determine the method's effectiveness. It should be noted that some of the conclusions from those studies can be misleading since the in-situ moisture susceptibility of asphalt mixtures is not just controlled by mixture properties. Drainage, roadway geometry, asphalt concrete density, construction variables, and production variability also control the moisture resistance. For this reason, studies directly comparing TSR values to field moisture resistance to evaluate the effectiveness of AASHTO T 283 (2014) (or any other methods) can be misleading.

Laboratory investigation

1. Although the average TSR values are higher for the mixtures with lime and Evotherm, the impact of additive usage on TSR values is not high. However, it should be noted that additives are not always the only (or major) factor controlling the moisture susceptibility of asphalt mixtures. Other mixture variables (such as asphalt

binder content, RAP content, binder type, etc.) also control moisture resistance. Through the statistical analysis, it was determined in this study that the impact of some of those major mixture variables (RAP content being the most significant) is masking the impact of those additives on TSR and moisture resistance. Increasing RAP content results in lower TSR values and potentially lower moisture resistance. This result also suggested that it is not possible to achieve high moisture resistance by only using lime or a chemical additive. Selecting the correct mixture variables is also important for reducing moisture susceptibility.

2. Based on the findings described in Conclusion 6 above regarding the negative impact of higher RAP on moisture susceptibility, the use of chemical additives to improve the stripping resistance of high RAP mixtures should be evaluated in a future research study.

3. None of the mixture variables are affecting the TSR-MIST2 values. This result points out that MIST2 conditioning may not be providing TSR test results that reflect the impact of different mixture properties (including additives) on the moisture susceptibility. This conclusion might be a result of the aggressive moisture conditioning resulting in highly variable damage in the asphalt mixture microstructure.

4. M5 and M6 were produced at the same plant and had the exact same ingredients except the additional antistripping (ARR) additive used in M5. Due to the presence of liquid antistrip in M5, it was expected to have a higher TSR value (meaning higher moisture resistance). However, according to the TSR values for vacuum conditioned M5 and M6 samples, M6 had a higher TSR value. However, this discrepancy might be a result of the variability observed during the plant mixture production.

5. Most of the plant-measured TSR values are close to the TSR values measured at the OSU-AMaP laboratory.

6. Resilient modulus testing is not providing any information significantly different from the TSR values. Since resilient modulus testing requires an expensive hydraulic or pneumatic test system, it is not recommended for moisture susceptibility testing in this study.

7. CT-Index or FI parameters that are related to the flexibility and cracking resistance of the asphalt mixtures are not recommended to be used to evaluate the moisture susceptibility of asphalt mixtures. The CT-Index parameters calculated from IDT test results might not be capturing the moisture susceptibility of the asphalt mixtures, while the strength parameter (which is also the parameter currently used for moisture susceptibility evaluation in AASHTO T 283, 2014) can provide a more reasonable output correlated with moisture susceptibility.

8. The B1/B2 ratios from the HWTT test specimens conditioned with vacuum conditioning are highly correlated with the TSR-Vacuum results while no significant correlation was observed when the HWTT results from unconditioned cores are used.

These results point out that submerging and testing HWTT specimens in water does not create internal damage equivalent to Vacuum saturation. For this reason, exposing the specimens to vacuum conditioning resulted in HWTT B1/B2 ratios that are highly correlated with the TSR results. This conclusion also points out the potential issues with implementing HWTT as a moisture susceptibility test without any prior conditioning.

9. None of the asphalt mixtures evaluated in this study show 3rd stage HWTT failure with a stripping inflection point (SIP) at the standard 50°C test temperature. Even when the HWTT temperatures were increased to 55°C and 60 °C, only Mix 4 provided a visible tertiary failure and a SIP. This conclusion is a result of the high rut resistance of the ODOT asphalt mixtures. Since HWTT results are mostly controlled by the rut resistance of the asphalt mixtures, it is not possible to isolate the moisture susceptibility-related portion of the deformation without conducting unsubmerged HWTD tests (See conclusions 4 and 5 above for findings from the literature supporting this conclusion). In the light of this conclusion and the literature review, SIP and stripping slope are not recommended to be used to quantify the moisture susceptibility of ODOT asphalt mixtures.

10. Although the colorimeter readings from the split (broken) IDT specimens were not found to be correlated with the TSR values of the same samples, those readings were able to clearly separate the mixes with and without additives. As described in Section 4.6.2.1, TSR results are highly affected by the mixture variables (binder content, RAP content, D-to-B ratio, etc.). In addition, TSR values are expected to be affected by both the cohesion and adhesion-related damage, while the colorimeter parameter L*RB from the broken cores are expected to represent only the adhesion-related failures (separation of binder from the aggregates). However, since most of the moisture-related failures in Oregon result from adhesion-related failures, L*RB values are expected to complement the TSR results and provide a quick and easy method to determine adhesion-related stripping in fractured samples. The repeatability and reliability of using the L*RB parameter for moisture susceptibility evaluation should be further explored in a future research study with cores extracted from pavements with known historical performance against moisture.

11. It is hypothesized that standard HWTD tests conducted in water simulate both moisture and rutting-related deformation. To isolate the moisture-related failure from the moisture and rutting-related failure, Lu (2005) suggested conducting HWTD tests in both dry and wet conditions. Since dry tests are expected to only simulate rutting resistance-related failures, subtracting the area under the average dry test rutting curves from the area under the wet test rutting curves is expected to provide a parameter (called ΔA_{RD}) correlated with the moisture susceptibility of the asphalt mixture (higher numbers being more susceptible).

    In this study, variable results were obtained regarding the effectiveness of the ΔA_{RD} parameter for moisture susceptibility evaluation. Based on the results, it is not possible to confidently conclude that the ΔA_{RD} parameter is capturing the impact of additives on moisture susceptibility. However, as also previously discussed, the use of additives may not always improve the moisture resistance due to problems with the mix design parameters or variability and issues during production. For this reason, in a future study, these results should be compared with the in-situ long-term moisture susceptibility of pavements constructed with these 6 mixes.

12. In this study, a special sensor system was used to determine the effectiveness of these sensors for moisture infiltration monitoring. The merit of this test lies in its ease of use and operation. Although the test is destructive in nature, it does not require the removal of full-depth cores. It can be readily applied in the field. The recordings are displayed in real-time on the cellphone screen via Bluetooth connection and a mobile application. If the holes are drilled properly and the caps of the sensors are sealed securely, this test has the potential to be used as an alternative to the traditional permeability tests and destructure forensic investigations for moisture damage monitoring.

13. In this study, it was proven that the electrical resistivity, a parameter commonly used for concrete porosity and durability evaluation, for asphalt mixtures could be measured with a high level of repeatability. The electrical resistivity measurements can capture the reduction in porosity as a result of the higher VFA. This result shows that electrical resistivity measurements for asphalt mixture porosity evaluation can be a promising method. However, additional research with several other mixture types and density levels must be conducted before adapting this test method for porosity and moisture susceptibility evaluation.

5.2 RECOMMENDATIONS

Based on the comprehensive literature review and the results of the laboratory investigations, this study recommends the use of a colorimeter in conjunction with the current AASHTO T 283 (2014) method to determine the adhesion and cohesion-related moisture susceptibility. According to the laboratory test results, vacuum saturation is able to create significant moisture damage in the asphalt microstructure, and no other conditioning method needs to be adapted to replace the vacuum saturation method. Colorimeter readings from the split AASHTO T 283 (2014) cores should be used to evaluate the impact of different additives on adhesion-related moisture resistance, while the TSR values can be used to evaluate both the adhesion and cohesion-related failures (although the TSR results are also affected by other mixture variables).

Based on the results of this study, HWTT is not recommended for moisture susceptibility evaluation.

5.3 SUGGESTED FUTURE RESEARCH

Suggestions for potential future research are provided below:

1. In this study, only PMLC specimens were used for laboratory testing. The effectiveness of the recommended colorimeter measurement method should be further validated by using laboratory mixed-laboratory compacted specimens (LMLC). Mixtures should be prepared with several different additives, RAP contents,

aggregate types, binder types, and binder contents. Asphalt binders and aggregate types with a poor agreement (due to similar electrical charges for aggregates and binders. See Section 2.2.2.3) should also be mixed and tested to determine the effectiveness of the proposed method in identifying the asphalt mixtures with poor moisture resistance.

2. Although the colorimeter CM-600D used in this study was able to separate the mixtures with additives from those with no additives, the high cost of the system (about $9,000) might make it harder to implement as an adhesion performance measurement tool. For this reason, the correlation of the measurements from the CM-600D model with the measurements from a lower-cost option (CM-23D with a price of about $5,000) should be evaluated in a future study. It is likely that the CM-23D model is also capable of capturing the stripping-related color change on the broken asphalt core faces at a lower cost.

3. The findings of this research study should be validated by conducting forensic investigations on the roadway sections constructed with the asphalt materials used for laboratory investigation in this research study. In the future, cores should be removed from the sections with cracking failures to inspect them for mixture-level moisture failures. Based on the findings, the effectiveness of AASHTO T 283 (2014) with the recommended colorimeter adaptation can be validated at the field level.

4. Once the proposed methodology is validated, the effectiveness of lime and several different anti-strip agents in mitigating moisture-related failures should be investigated.

5. Hicks (1991) indicated that sharp aggregate edges could rupture the asphalt binder film around the highly angular aggregates and result in an asphalt mixture with poor moisture resistance. Water can easily seep through the broken asphalt binder film and fill the zone between the binder and the aggregate. With the application of heavy vehicular loads, high pore pressures around this zone can separate the aggregates from the binder and result in severe stripping. The impact of crushed aggregates with more cubical shapes on moisture susceptibility should be investigated in a future study.

6. A limited preliminary study was conducted in this research project to evaluate the effectiveness of a moisture sensor technology for permeability and infiltration identification and measurement. In a future study, the effectiveness of this sensor technology should be investigated by conducting several experiments in the laboratory and field. The sensors can be installed on a highway section at various locations. At different times of the year (before, after, and during the rainy seasons), the moisture level can be measured by wirelessly connecting to the sensors from the side of the road (without needing a traffic closure). In this way, without conducting destructive forensic investigations (coring or trenching), moisture-related failures at the structural level can easily be explained.

7. The successful measurement of the electrical resistivity of the asphalt cores is promising and can provide a major breakthrough in determining the pore interconnectivity of asphalt materials. However, the experimental factorial followed in this study was limited and included only a few mixture types. In a future research study, a greater number of replicates and samples with different air void contents and aggregate gradations should be used to determine the effectiveness and accuracy of the test. Results should also be checked towards more accurate (ground truth) methods such as X-Ray CT image data to validate the effectiveness and reliability of the measured parameters.

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