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Evaluation Of Balanced Mix Design Gyrations (Ndesign) For North Dakota's Lower Class HMA Pavement

Anjo Maurice Mate

EVALUATION OF BALANCED MIX DESIGN GYRATIONS (N_{design}) FOR NORTH DAKOTA'S LOWER CLASS HMA PAVEMENT

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Anjo Maurice Mate
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Name: Anjo Maurice Mate

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This document, submitted in partial fulfillment of the requirements for the degree from the University of North Dakota, has been read by the Faculty Advisory Committee under whom the work has been done and is hereby approved.

DocuSigned by:
Nabil Suleiman

Nabil Suleiman Ph.D, Chairperson., Associate Professor

DocuSigned by:
Dr. Daba Gedafa

Daba Gedafa Ph.D, Associate Professor

DocuSigned by:
Iraj Mamaghani

Iraj Mamaghani Ph.D, Associate Professor

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DocuSigned by:
Chris Nelson

Chris Nelson
Dean of the School of Graduate Studies

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Anjo Maurice Mate
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ABSTRACT

The number of Superpave mix design of gyrations (N_{design}) has been adopted by North Dakota Department of Transportation (NDDOT) to produce mixes of same density representing the field conditions for a specified amount of traffic. As Superpave mix design was developed for high volume roads, various research suggests that there is a need to develop a new mix design criterion for medium and low volume traffic. For low volume roads durability performance is generally affected by the environment and not the traffic volume. Some researchers notice that the N_{design} of 75 gyrations for a 20-year traffic loading (in millions of ESALs) between that 0.3 and 3 are too high and needs to be reduced. High N_{design} numbers tend to lower the asphalt binder, thus lower the durability of asphalt mix. In contrast, if N_{design} is reduced it tends to increase the asphalt binder, thus increase the durability of asphalt mix.

The main objective of this study is to determine the effect of varying number of design gyrations on hot mix asphalt (HMA) performances in terms of low-temperature cracking (LTC), fatigue cracking (FC), and rutting distresses. Project mixes that were chosen were constructed based on N_{design} values of 75, 65, and 55 or 50 gyrations.

Test results showed that the higher number of gyrations with less asphalt the lower fatigue cracking and lower low-temperature cracking resistance. In contrast, the mixes with lower number of gyrations with higher asphalt contents showed higher resistance to cracking. Also, results showed that PG 58H-28 had slightly better fracture

energies than PG 58H-34. Lastly, lab mixes had better fracture energies as compared to field mixes.

Chapter 1

INTRODUCTION

1.1 Background

North Dakota is one of the coldest states based on average temperature, which is 40.4 degrees Fahrenheit. The coldest temperature in the area ever recorded was -60 degrees Fahrenheit in 1936. On the other hand, summer temperatures exceed 100 degrees Fahrenheit. Due to these intense conditions, Asphalt pavements in the area tend to be prone to rutting, fatigue cracking and thermal cracking.

1.2 Major Asphalt Distresses in North Dakota

1.2.1 Rutting.

Rutting is defined as the permanent deformation or consolidation that accumulates in the asphalt (Liley, 2018). Ruts can be usually seen during summer where the temperature is high, and when the binder on the surface of older asphalt roads begin to stick to the bottom of the shoes. It occurs because the aggregate and binder in asphalt roads can move. Ruts are visible after rain when they are filled with water (Washington Asphalt Pavement Association, 2010). Additionally, ruts can also be formed when a truck drives over a road that has lack of internal strength to resist permanent deformation under stress imposed by the loaded wheel of the vehicle tires (Liley, 2018). If this distress is not treated, it can cause accidents to drivers and passengers. Figures 1 and 2 show a typical rutting distresses in asphalt pavement.

Figure 1. Rutting in a two-lane Asphalt Pavement (Pavement Interactive, 2009)

Figure 2. Rutting on side curb of Asphalt Pavement (Pavement Interactive, 2009)

There are two kinds of rutting, mix rutting and subgrade rutting (Washington Asphalt Pavement Association, 2010). Mix rutting is when the subgrade does not rut yet, but the pavement surface shows wheel path depressions from compaction or mix design problems. In contrast, subgrade rutting, occurs when the subgrade already has exhibited wheel path depressions due to vehicle loading, which makes the asphalt pavement more consolidated under the action of traffic (Pavement Interactive 2009; Ohio Asphalt, 2004). Hydroplaning is a phenomenon caused by ruts filled with water as vehicle skid resistance is reduced closed to zero. It may be hazardous to drivers as it tends to pull a vehicle towards the rut path as it is steered across the rut which may cause vehicle collisions (Washington Asphalt Pavement Association 2010; Liley, 2018). Figure 3 shows that the pavement is displaced under the tires and humps up outside the wheel tracks. Users will not notice the depressions, but the depression will slowly pull the car which may cause fatal injuries or accidents to users. Due to displacement under the tires, it will hump up outside the wheel tracks.

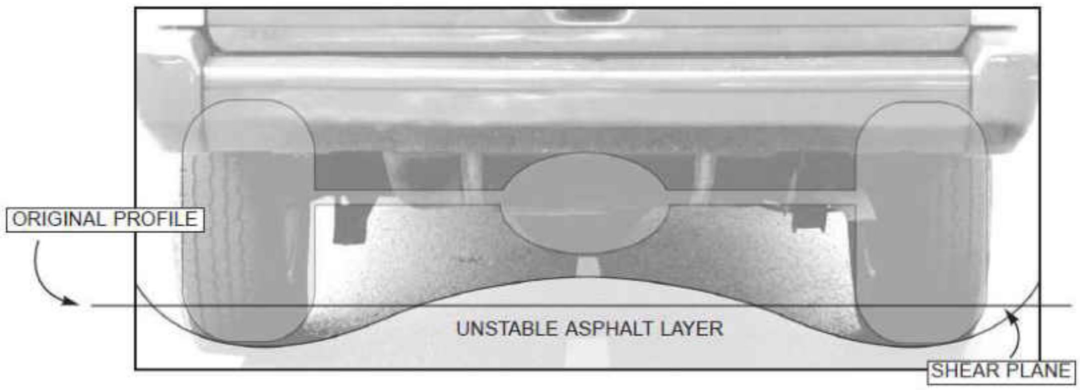


Figure 3. Asphalt Pavement displaced under the tires (Ohio Asphalt, 2004)

Lack of compaction would be a probable cause of rutting in asphalt pavement. According to Liley (2018) one of probable cause would be insufficient thickness of asphalt pavement and weak asphalt mixtures. Liley (2018) further explains that asphalt pavement requires specification that would be constructed in a way to prevent rutting and other distresses. Additionally, Ohio Asphalt (2004) added that mix with lack of internal strength to resist deformations under loaded tires will cause rutting. Internal strength is affected by friction characteristics of the aggregates, especially the fine aggregate (Ohio Asphalt, 2004). This kind of distress has lots of various issues and it can be prevented by mixtures that are properly designed.

However, to reduce rutting failure to asphalt pavement, ruts can be investigated, and prevented. Ohio Asphalt (2004) claims that ruts can be prevented by keeping it to a stiffer, stronger subbase in reducing the chances of rutting. Subbase is explained to be important in the road system as it provides the support for which the road is built on. For construction of good quality rutting resistance, monitoring for quality control is a must. Also, administration of weight, and number of passes of the roller over a section of asphalt play a major role in ensuring the quality of asphalt surface. Using angular aggregates tends to have higher internal friction that helps resist deformation under heavy loads (Ohio Asphalt, 2004). Liley (2018) argues that another way to combat rutting is to add more fine aggregate to increase its friction within the mix. Some suggestions would be using a GPS system or sensors in the roller to make sure that the roller can keep track of the number of passes. GPS or sensors can be utilized because traditional method sometimes missed sections resulting in roads not receiving proper compaction.

1.2.2 Fatigue Cracking.

Fatigue or alligator cracking is one of the major asphalt distress issues in North Dakota. It is a series of interconnected cracks caused by fatigue failure of hot mix asphalt under repetition of vehicle loadings (West et al, 2018). For thinner pavements, cracking starts at the bottom of the HMA layer where tensile stress is the highest and then proliferate towards the surface as one or more longitudinal cracks which is called bottom up or classic fatigue cracking. In contrast, thicker pavements essentially start from the top in areas of high localized tensile stress resulting from tire to pavement interaction and asphalt binder aging. After the said repeated loadings, longitudinal cracks connect forming many sided sharp angled pieces that turns into a pattern resembling the back of an alligator or crocodile (Washington Asphalt Pavement Association 2010). Furthermore, West et al. (2018) added that fatigue cracking is also affected by aging that correlates with embrittlement of asphalt binder. Due to continuous loading and climate factors to asphalt pavement, it reduces its structural integrity causing it to crack and may become a pothole that would risk its users. Figure 4, as shown, will start as a crack and propagate looking like a back of an alligator.

Figure 4. Images of Fatigue Cracking in Asphalt Pavement (Pavement Interactive, 2009)

There are several possible causes of fatigue cracking. Inadequate structural support, which can be caused by various issues like mix gradation problems. Decrease in pavement load supporting characteristics, like loss of base, subbase or subgrade support. Stripping on the bottom of the HMA layer, which contributes little to pavement strength so the effective HMA thickness decreases. Also, due to additional loads in traffic, asphalt pavement with poor construction and inadequate structural design, will fail and cause to crack and form alligator cracks on the surface (Washington Asphalt Pavement Association, 2010). As different problems may arise that will cause fatigue cracking, it is good practice to prevent or investigate the problem before the pavement loses its structural integrity.

Repair of fatigue cracking should be investigated to determine the cause of failure. Washington Asphalt Pavement Association (2010) explains that if an alligator pattern is demonstrated by the pavement, repair by crack sealing is ineffective. Investigation of the asphalt must be done comprising of digging a pit or coring in the asphalt pavement to determine the pavement's structural makeup as well if subsurface moisture is a factor. If the crack is small, it might be an indication of a loss of subgrade support. In contrast, if there is a huge crack, it is an indication of a general structural failure. HMA overlay that is structurally strong to carry heavy loads over the entire pavement surface is a solution. Prevention of fatigue cracking is attainable if the design and construction of asphalt pavement can support the expected traffic loads of a given highway.

1.2.3 Thermal Cracking.

Thermal or transverse cracking is the distress that is found in asphalt pavements in low temperature climates. Transverse cracking is a common problem and a safety hazard because the roads are constantly in use. Cracks develop when temperatures drop, and the asphalt pavement shrinks and contracts. This is the reason why it is also referred to as thermal cracking (Bradshaw, 2016). As the asphalt begins to tighten, tensile stress builds up to a critical point at which cracks are formed (Minnesota Department of Transportation, 2014). Aschenbrener (1995) added that transverse cracks are relatively perpendicular to the centerline of the pavement. Cracks start usually on the surface of the pavement and then gradually sink deeper below the surface like the figure 5 shown below.

Figure 5. Images of Thermal Cracking in Asphalt Pavement (Pavement Interactive, 2009)

Transverse cracking can commence by single low temperature event or by multiple warming and cooling cycles and then multiply by further low temperature or traffic loadings (Minnesota Department of Transportation, 2014). Aschenbrener (1995) and Bradshaw (2016) studied that heavy snow and rain can cause the cracks to erode more aggressively over time, water can enter cracks and cause raveling of the joint and/or loss of base support. Investigation of thermal cracking is quantified by the frequency or spacing of the cracks and crack width (Aschenbrener, 1995). Due to thermal cracking, decrease in rideability of asphalt pavement is expected if not treated or investigated.

Testing of asphalt mixtures is important to accurately predict low temperature cracking performance of asphalt pavement in the field. Testing includes sophisticated techniques based on fracture mechanics rather than the current practice of stiffness and strength testing (Minnesota Department of Transportation, 2014). If cracks are left untreated for too long, they can lead to more problems and potentially more expensive repairs in the long run (Bradshaw, 2016). A research at Iowa revealed that cracks that were sealed properly are not as badly deteriorated as those which have not been sealed (Shelquist, et al. 1981). Thus, it was recommended by Aschenbrener (1995) that material properties can increase resistance of thermal cracking. Additionally, it was recommended by Shelquist, et al. (1981) that adopting a positive procedure requiring timely sealing of cracks is needed and strengthening specifications for preparing pavement surfaces for asphalt overlays is a must. Therefore, with pavement management and proper testing materials thermal cracks can be treated or prevented.

1.3 Problem Statement

In the past few decades, North Dakota is focusing on rutting failure in asphalt pavement. Concentrating too much on rutting failure makes the compaction effort (N_{design}) of Superpave mixes maintained at 75 gyrations for all pavement classes regardless of traffic level. According to North Dakota Department of Transportation (NDDOT) engineers and materials coordinators rutting was always in check in the state.

Recently, NDDOT experts started to recognize that the rut resistant pavements constructed were failing because they were dry, brittle, and in some cases have various permeability problems because of density issues. Durability in asphalt pavement needs to be addressed suggested by some engineers from the state. The initial solution is by lowering the number of gyrations allowing binder contents to increase while aggregate gradation is maintained. Based from NDDOT there are districts that attempted to reduce N_{design} on projects from 75 to 65 or even 50 gyrations on lower volume roads where rutting was not a concern. District engineers noticed that the binder content increased by 0.1 to 0.2 percent, which was expected to help with durability issues in asphalt pavement.

Low temperature and fatigue cracking and other durability related modes of failure are the root of damages in asphalt pavement here in North Dakota's lower pavement classifications. Lowering the number of gyrations, while keeping aggregate gradations the same will probably be the solution to this dilemma. Conclusively, this will lead to an increase in binder content, voids in mineral aggregate, and film thickness, but surely, this will help durability issues here in North Dakota.

1.4 Objective

The main objectives of this research study are the following:

a) Determine the effect of reducing the number of design gyrations on HMA performance of various pavement classes in terms of rutting, low-temperature cracking (LTC), and fatigue cracking (FC).

b) Investigate the effect of reduced number of gyrations based on three N_{design} values (levels) of 75, 65 and either 55 or 50 gyrations.

c) Develop an appropriate number of gyrations (N_{design}) that will produce balance mix designs that will be recommended for various pavement classes based on their tested performances.

1.5 Organization of Thesis

Chapter I introduces the history, objectives, problem statement and major asphalt pavement distresses in North Dakota. Chapter II is about literature review and calibration efforts of N_{design} by various researchers. Chapter III describes the methods used in this study. Chapter IV shows the results and its discussion. Chapter V presents conclusion, recommendations and future research.

Chapter 2

LITERATURE REVIEW

2.1 History of Balanced Mix Design

In early 1900s, the Superior Performing Asphalt Pavement System (Superpave) mix design method was developed from the Strategic Highway Research Program (SHRP). The primary focus of Superpave is to limit detrimental distresses of asphalt pavements. It takes account of the changes in environmental conditions, traffic loading, and axle configurations. Additionally, Superpave assesses asphalt binder, aggregate properties, mixture analysis, and volumetric properties in HMA (Williams, et. al., 2016). Volumetric analysis of HMAs is mainly used to determine optimum asphalt content in the mixture. Superpave gyratory compactor (SGC) is generally the compaction device used to compact laboratory specimens. As it is heavily dependent on traffic levels, it is generally expressed as 18,000 lbs ESALs (Williams, et. al., 2016). HMA samples are generally compacted in an internal angle of gyration of 1.16° (external angle 1.25°) with a constant pressure of 600 kilopascal (kPa) (Prowell & Brown, 2007). National Cooperative Highway Research Program (NCHRP) recommended compaction effort for different levels and was denoted as $N_{initial}$, N_{design}, and N_{max} as shown in table 1. $N_{initial}$ is the number of gyrations which indicates tender mix during compaction which is undesirable in the field. N_{design}, is the design number of gyrations to make sample with same density that is expected in the field which generally has 4% air voids. Finally, N_{max} is the number of gyrations required to produce at the laboratory density that must be never exceeded in the field (Roberts, et. al., 2002).

Table 1. NCHRP Compaction Parameters (NCHRP, 2001)

Design ESALs (millions)	Compaction Parameter		
	$N_{initial}$	N_{design}	N_{max}
< 0.3	6	50	75
0.3 to < 3	7	75	115
3 to < 30	8	100	160
≥ 30	9	125	205

In 2015, Federal Highway Agency (FHWA) Expert Task Group on Mixtures and Construction formed a Balanced Mix Design (BMD) Task Force. The objective of the BMD group was to assess "asphalt mix design using performance tests on appropriately conditioned specimens that address multiple modes of distress taking into consideration mix aging, traffic, climate and location within the pavement structure." (National Center for Asphalt Technology, 2017). The said task force was able to identify three potential approaches to the use of Balance Mix Design; the approaches are schematically illustrated by the flowchart displayed in figure 6 and briefly discussed as follows:

1. **Volumetric Design with Performance Verification** is a straight Superpave volumetric mix design approach with performance tests operated at the end. Basically, if the mixture does not pass performance tests, the entire design process is repeated. Currently used in Illinois, Louisiana, New Jersey, Texas and Wisconsin. (National Center for Asphalt Technology, 2017)
2. **Performance-Modified Volumetric Mix Design** is the approach that begins with the Superpave Mix design method to build an initial aggregate blend and asphalt content. Adjusting Mix proportions is granted to meet performance tests. Currently, California uses this approach.

3. The **Performance Design** approach starts with evaluation of mix trials using performance tests. Minimum requirements may be set for asphalt binders and aggregate properties. Volumetric criteria may be used as non-mandatory guides but not as design criteria. This approach is not currently used. (National Center for Asphalt Technology, 2017)

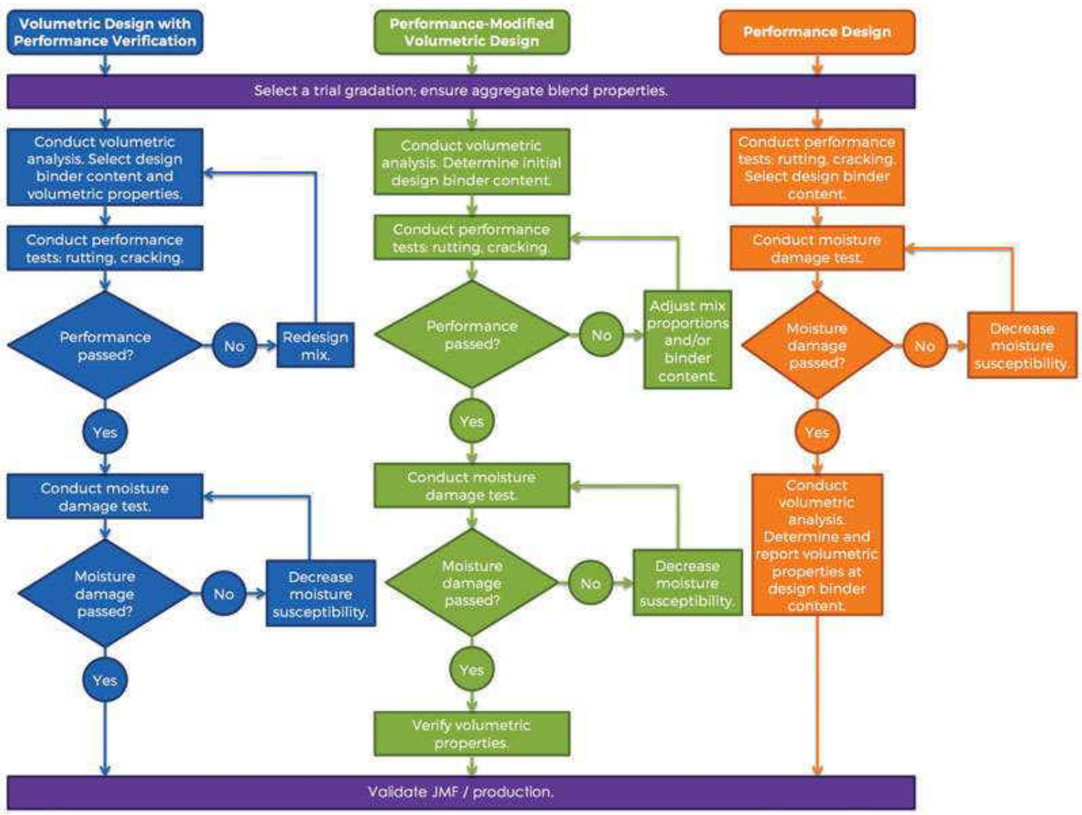


Figure 6. Flow Chart: Balanced Mix Design Approaches (NCAT 2017)

There are numerous performance tests that were developed over the past few decades by researchers who assessed the rutting resistance, cracking resistance, and moistures susceptibility of asphalt mixtures. Recognizing the different mechanisms in crack initiation, cracking tests can be more categorized into thermal cracking, reflection cracking, bottom-up fatigue cracking, and top-down fatigue cracking. Table 2 provides a list of mixture performance tests that are commonly used in Asphalt research and being recognized by highway agencies for use in mix design. (NCAT 2017). The different types of performance tests in asphalt pavement will ensure the durability of asphalt in various types of cracks or distresses.

Table 2. Common Mixture Asphalt Tests (NCAT 2017)

Mixture Property	Laboratory Test	Test Standard
Thermal Cracking	Disk-Shaped Compact Tension Test	ASTM D7313-13
	Indirect Tensile (IDT) Test	AASHTO T 322-07
	Semi-Circular Bend (SCB) Test	AASHTO TP 105-13
	Thermal Stress Restrained Specimen Test	BS EN12697-4
Reflection Cracking	Disk-Shaped Compact Tension Test	ASTM D7313-13
	Texas Overlay Test	TxDOT Tex-248-F NJDOT B-10
Bottom-Up Fatigue Cracking	Direct Tension Cyclic Fatigue Test	AASHTO TP 107-14
	Flexural Bending Beam Fatigue Test	AASHTO T 321 ASTM D7460
	IDT Fracture Energy Test	N/A
	Illinois Flexibility Index Test	AASHTO TP 124-16
	SCB at Intermediate Temperature	LaDOTD TR 330-14 ASTM D8044-16
	Texas Overlay Test	TxDOT Tex-248-F
Top-Down Fatigue Cracking	Direct Tension Test	N/A
	IDT Energy Ratio Test	N/A
Rutting	Asphalt Pavement Analyzer	AASHTO T 340
	Flow Number	AASHTO TP 79-15
	Hamburg Wheel Tracking Test	AASHTO T 324
	Superpave Shear Tester	AASHTO T 320-07
	Triaxial Stress Sweep Test	AASHTO TP 116-15
Moisture Susceptibility	Hamburg Wheel Tracking Test	AASHTO T 324
	Tensile Strength Ratio	AASHTO T 283

2.2 Calibration Efforts of N_{design}

The design number of gyrations (N_{design}) was introduced by the Strategic Highway Research Program and is used in the Superpave mix design method, which has been commonly used for HMA design throughout the nation since 1996. As the N_{design} is used to simulate field compaction during the construction there have been reports that it produced air voids that are unable to reach ultimate pavement density within the initial 2 to 3 years of post-construction, potentially impacting long term performance of HMAs. Regarding durability problems of asphalt pavement, there had been various research to investigate the current levels with existing mixes and did recommendations to calibrate or identify the optimum N_{design} with the use of performance tests.

Aguilar-Moya et al. (2001) established that the number of design gyrations using the Superpave Gyratory Compactor could be reduced significantly to optimize fatigue life of the asphalt mixes. The study used the relative performance base approach as applied to two performance-related tests such as the Hamburg Wheel Tracking Device (HWTD) and the four-point bending beam. It was observed that in 100 gyrations which is the current specification for 1 million to 30 million ESALs, the rutting performance in the laboratory was very high (75% to 95% relative performance); however, the fatigue performance is typically low (varying from 20% to 70% relative performance). Aguilar-Moya et al. (2001) saw from the average relative performance of all the mixes tested, it was found that generally 55 to 85 gyrations on the SGC optimize the performance of the asphalt mixes. However, these recommendations were based on limited sample mixes.

Brown & Mallick (1998) conducted research in which specimens were compacted using the Superpave Gyratory Compactor with different gyration levels and then were compared with the density of in-place cores obtained from pavement sections at various levels of traffic. They took the cores immediately after construction and after one, two and three years of service. Results from the study concluded that, gyrations required to achieve the one and two year in-place density were below 100 gyrations for all mixtures (Brown & Mallick, 1998). Second, N_{design} gyration may have been too high for low traffic volume roadways but, needed to be further evaluated in the future after three years of recorded in-density. Finally, N_{desgin} values obtained were approximately 30 gyrations lower than those specified under the Superpave (Brown & Mallick, 1998).

Prowell & Brown (2007) conducted research to verify N_{design} levels to optimize field performance. They tested several cores via mobile laboratory and brought the gyratory specimens and loose mix to NCAT for testing and to also determine Maximum Specific Gravity of the mix (G_{mm}), asphalt content and gradation. It was discussed in the research that the ultimate density was critical to the performance of HMA pavement (Prowell & Brown, 2007). Furthermore, higher asphalt contents for a given aggregate structure were generally easier to compact. However, if the laboratory compaction was too high, it could have been difficult to achieve the required as-constructed density in the field. Prowell & Brown (2007) concluded that asphalt pavements appear to reach their density after 2 years of traffic. Also, high performance grade (PG) and the high temperature bumps between the climatic and specified PG were found to significantly affect pavement densification, with stiffer binders resulting in less densification. Also, number of gyrations to match ultimate in-place density was calculated, the values for two compactors used in the study differed by approximately 20 gyrations. Finally, several analyses were conducted to evaluate the N_{design} levels and indicated that the N_{design} levels could be reduced.

Qarouach (2013) investigated the effect of N_{design} values on performance of Superpave mixtures. The objectives of the study were to evaluate the sensitivity of asphalt volumetric properties to different design levels, investigate the effects of changes of N_{design} values on mixtures, and to recommend N_{design} levels for different North Carolina Department of Transportation (NCDOT) mixtures varying traffic and reliability levels.

Based from analysis and results, it was noticed that as N_{design} increases, the resistance of a mix to rutting increases whereas fatigue resistance decreases due to lower asphalt content required to achieve the target air void content of 4%. Additionally, Superpave mix designs of all surface mixes were performed to determine the optimum asphalt content corresponding to the four N_{design} levels. Furthermore, it showed that the optimum asphalt content decreased with an increase in N_{design} level. The findings reinforce the theoretical basis for this study that using a higher N_{design} for mix design requires lower binder content and results in stiffer mix.

Mercado (2015) validated the current N_{design} levels for one-hundred thousand (100k) to ten million (10M) ESALs surface mix designs. The researcher assessed the compactability of mixes under the current mix design procedures by using the calculated gyratory slope from quality control and quality assurance and to provide N_{design} recommendations for the laboratory Superpave mix designs to the Iowa Department of Transportation (IDOT). The field cores were randomly selected from different projects around the state provided by the Iowa DOT. The laboratory testing was conducted using the American Society for Testing and Materials (ASTM) and American Association of State Highways and Transportation Officials (AASHTO) standards. Mercado (2015) concluded that based on laboratory testing the current N_{design} table in Superpave Mix Design is possibly too high. Thus, if the N_{design} is high it may result to durability issues especially with the low temperature and fatigue cracking.

2.3 Low Volume Roads N_{Design} Calibration Effort

Low volume roads (LVRs) are defined as roads lying outside of built-up areas of cities, towns, and communities and shall have an Equivalent Single Axle Loading of less than 300,000. To increase durability and longevity of LVR asphalt pavements various research efforts were made to calibrate N_{design} for low volume roads.

Cross & Choho Lee (2000) evaluated void properties at the original and revised N_{design} gyrations and the effect of reducing the ram pressure from 600 kPa to 400 kPa. Superpave Gyratory Compactor (SGC) was used to compact the field mix to establish the number of gyrations required to reach field density. They wanted to prove that N_{design} values were inaccurate for all levels of traffic. It was found that N_{design} was developed with higher quality aggregates that were typically found in Kansas and the Midwest (Cross & Choho Lee, 2000). The primary problem in meeting the Superpave Level 1 mix requirements has typically been Voids in Mineral Aggregates (VMA), which is explained as a volumetric property and a function of compactive effort. They utilized the Asphalt Pavement Analyzer (APA) with traffic less than one million ESALs. As the result was evaluated, it was observed that the gyratory compaction effort was higher than 50-blow Marshall compaction. The use of SGC resulted in an average reduction in VMA of 1.2% to 1.9% when compared to 50-blow Marshall compaction. Thus, SGC resulted in an average reduction in optimum asphalt content of 0.5% to 0.8% when compared to Marshall compaction (Cross & Choho Lee, 2000). Also, reducing the ram pressure from 600 to 400 kPa had the same effect on the asphalt mix (Cross & Choho Lee, 2000). Cross & Choho Lee (2000) recommended that the effect of reducing the VMA requirement on durability of bituminous mixes should be evaluated. Also, it may be possible to reduce the VMA requirement by 0.5% to 1% without sacrificing the performance of low volume pavements.

Vitillo, et al., (2006) compared the composition (gradations and binder content), volumetric parameters, rutting, fatigue, permeability and average asphalt binder thickness of the new Superpave mixtures with those of the proven Marshall mixtures developed for low volume roads in New Jersey. Also, the research evaluated the number of gyrations needed to produce equivalent air voids to the Marshall mixtures in the Superpave Gyratory Compactor. For sampling preparation and testing, they used the specifications in accordance with AASHTO T-245 and AASHTO R35 for Marshall and Superpave respectively. Superpave mix design samples were compacted using $N_{ini} = 6$, $N_{Design} = 50$ and $N_{Max} = 75$ gyrations which is specified for design ESALs of less than 0.3 million. Vitillo, et al. (2006) evaluated that the Superpave design for low-volume roads provided a positive assessment. As they compared the two approaches, Superpave mixtures resulted in higher optimum asphalt binder content. The research evaluated that the number of gyrations for low-volume road design was found to be correlated to the bulk specific gravity of the aggregate blend (G_{sb}) and the maximum specific gravity of the bituminous mixture. Since the G_{sb} is typically known before determination of the optimum asphalt binder content, the G_{sb} potentially may be used to estimate the level of gyrations necessary to design well performing Superpave mixes in New Jersey (Vitillo, et al. 2006).

Mogawe & Mallick (2004) developed compaction and volumetric design criteria for designing asphalt mixes for low volume roads. They also evaluated the performance of mix design according to its criteria and provided recommendations for proper implementation of the new mix design system by the states DOTs. They set gyration numbers to 30, 40, 50 and 75. The highest gyration was 75 since it is being used by many

DOTs. The lowest number of 30 was suggested since lowering gyration level below 30 would result in abnormally high asphalt content (Mogawe & Mallick, 2004). After all tests were evaluated, they concluded that film thickness of 11 microns in samples compacted to 7% air voids was found to be desirable from considerations of stability and durability. N_{design} of 50 is recommended for compacting HMA for low volume roads in New England (Mogawe & Mallick, 2004). They also suggested that APA is a good proof testing equipment to evaluate rutting potential of asphalt. Furthermore, the research recommended that balancing asphalt content to suit durability and stability can be done by experienced engineers with local materials, climate and traffic. Finally, balancing can be made less critical by using polymer modified HMA which allows users to provide high asphalt content without increasing potential rutting.

2.4 Asphalt Performance Tests

2.4.1 Asphalt Pavement Analyzer Test (APA).

Asphalt pavement analyzer (APA) is a wheel tracking device that is used to run simulative test that measures HMA qualities by rolling a small loaded wheel device repeatedly across a mixed asphalt specimen. The APA is a second-generation device that was originally developed in the 1980’s as the Georgia Loaded Wheel tester (GLWT), which was a device designed for rut proof testing and field quality control. The primary purpose of GLWT was to perform efficient, effective and routine laboratory rut proof testing and field production quality control of HMA as shown in figure 7. Figure 8 is a modification of GLWT and was first made in 1996 by Pavement Technology, Inc. Since then, APA has been utilized to evaluate rutting, fatigue, and moisture resistance of HMA mixtures.

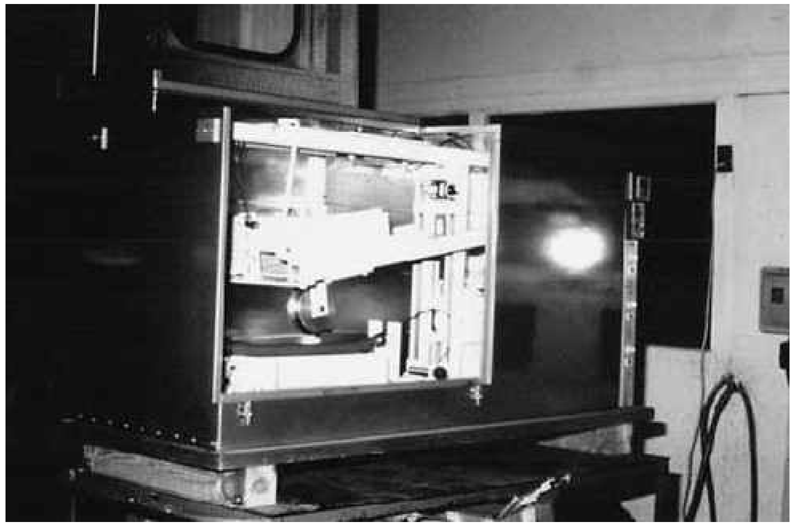


Figure 7. Georgia Loaded Wheel Tester (NCHRP Report 508, 2003)

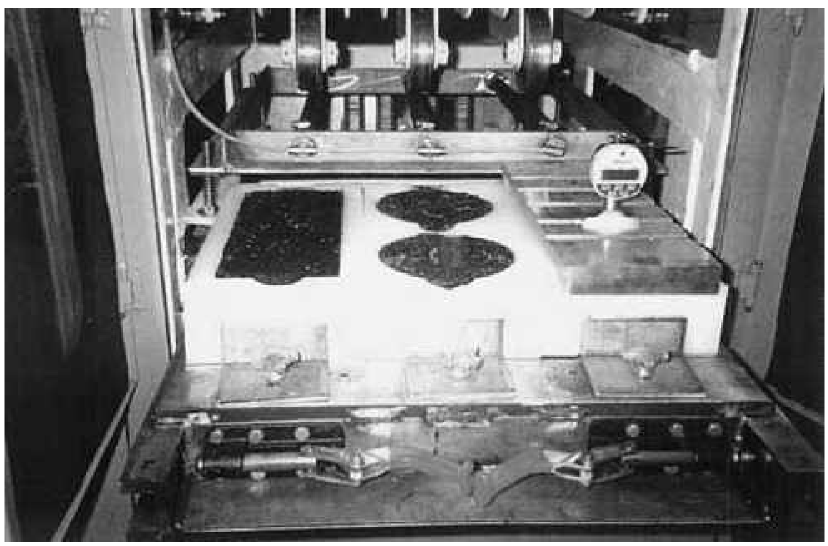


Figure 8. Asphalt Pavement Analyzer (NCHRP Report 508, 2003)

Kandhal & Mallick (1999), evaluated APA for HMA mix design. The objective of their study was to evaluate the asphalt pavement analyzer as a tool of evaluating rut potential of HMA with different aggregate gradations and asphalt binders. The test plan was to test mixes with different aggregates, gradation, nominal maximum size aggregates, asphalt binder, mix prepared with granite, limestone, and gravel. Tests were conducted under dry conditions with mixes obtained from high, intermediate and low rutting pavements. All APA tests were conducted with wheel load of 445 Newtons (N) and a hose pressure of 690 kPa. They explained that gradation of aggregates is the single most important property that determines the stability of a mix. Mixes with different aggregate gradations are likely to have different stability and different rutting potential. Additionally, they believed that the type of aggregate top size has significant effect on rutting potential. They also found out that APA is sensitive to aggregate gradation based on significance of differences in rut depths. Furthermore, it is found to be sensitive to the asphalt binder PG grade based on statistical significance difference in rut depths. Lastly, it was concluded that APA has a potential to accurately predict the rutting potential of hot mix asphalt mixes.

As Minnesota Department of Transportation (MNDOT) was looking to purchase APA, Skok, et. al. (2000) made an evaluation of APA. The purpose of their report was to determine if APA is a tool to evaluate the rutting susceptibility of Minnesota HMA. To determine if APA was a good tool for evaluating the rutting of asphalt, they developed a questionnaire and sent out to members of APA users' group. Majority of the responses indicated that most of the users were satisfied with the results and reliability of the APA. So, it was concluded that MNDOT must have the APA machine as it determines rutting and it is available with either American Society for Testing Materials (ASTM) or American Association of State Highway and Transportation Officials (AASHTO) formats.

2.4.2 Disk-Shaped Compaction Test (DCT).

Disk-shaped compaction test (DCT) is a fracture test that predicts fracture resistance of asphalt concrete from conventional engineering parameters, such as modulus and tensile strength. To fully understand the crack initiation and propagation in asphalt concrete, fracture mechanics must be understood to understand the evolution of performance-based pavement design. Fracture mechanics had been used since the early 1970's which was utilized to analyze the fracture behavior of concrete. Since then, DCT was used to evaluate the low temperature cracking of asphalt which is the most prevalent pavement distress especially in the cold climate areas (Wagoner, et. al., 2005).

Wagoner, et. al. (2005) described the development of a practical test to obtain the fracture energy of asphalt and field specimens. They found out that with the use of DCT geometry, it was considered a practical geometry that can be fabricated from cylindrical cores from in-place pavements or gyratory compacted specimens. The DCT geometry was developed using the ASTM E399 specification. They found out that DCT geometry, as shown in Figure 9, to be promising for obtaining the fracture energy of asphalt concrete that is amenable for both laboratory and field core specimens.

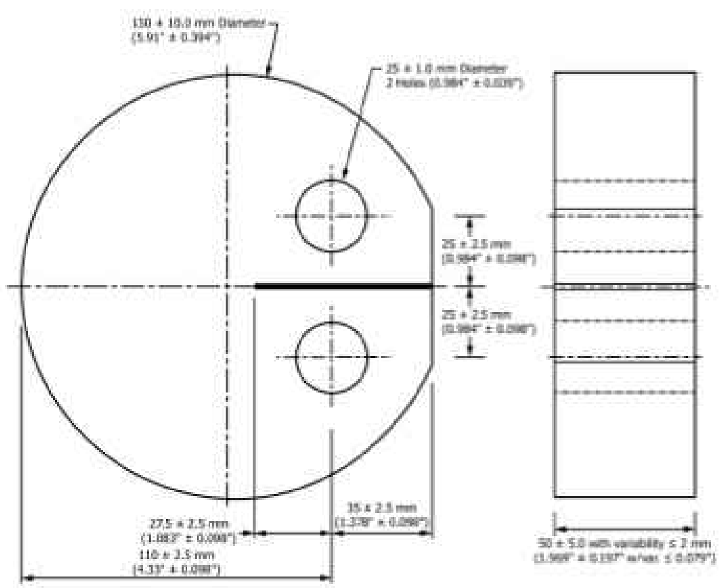


Figure 9. Disc-Shaped Compaction Test (DCT) Sample Dimension (Wagoner, et. al. 2005)

2.4.3 Semi-Circular Bending Test (SCB).

Semi-Circular Bending (SCB) test is another fracture test based on linear-elastic fracture mechanics (LEFM). The SCB was proposed by Chong and Kuruppu (2012) because they noticed that some tests were difficult to perform using rock materials. It was adopted by pavement engineers to understand fracture characteristics of different asphalt mixtures which led to the development of standard protocols for monotonic loading conditions. The SCB test has shown great potential research for determining the mixed mode fracture behavior of asphalt mixtures by simply adjusting the inclination angle of the notch or the space between two supports. Test specimens for SCB test is either made by SGC or taken from a core which was drilled from the field. The disc is cut from the gyratory sample or core, and this disc is sawn into two equal parts resulting in two semi-circular specimens as shown in the Figure 10.

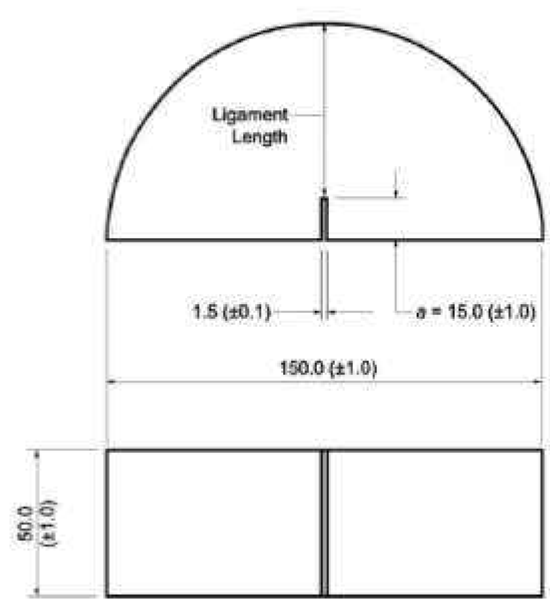


Figure 10. Semi Circular Bending (SCB) Sample Test Dimension (Nsengiyumva, et. al. 2015)

Nsengiyumva, et. al., (2015) examined reliability and practicality of SCB test for evaluating the fracture characteristics of asphalt concrete mixtures. They investigated

SCB for its repeatability for fracture test method by integrating a statistical experiment approach to identify testing variables of the SCB tests. After statistical analysis of 18 specimens with typical testing variables, it was found that five (5) to six (6) specimens were a reasonable sample size that could properly represent asphalt concrete fracture behavior using SCB. They also investigated the sensitivity of the SCB test using the previously determined testing variables. Asphalt mixtures were collected from 12 field construction projects in Nebraska and used as SCB specimens. They concluded that SCB test method is proved to be repeatable and sensitive to changes in mixture and thus a promising tool for evaluating the fatigue fracture resistance of AC mixtures.

Saha & Biligiri, (2012) compiled the current knowledge about the utilization of SCB test to evaluate fracture properties of HMA. There was limited research regarding SCB test but still it was contemplated that the methodology of the test turns out to be a promising candidate to assess fracture performance. A review made by the authors presented the state-of-the-art utilization of SCB test to evaluate fracture properties of different asphalt mixtures. Furthermore, the study focused on the fundamental assessment of fractures through the static SCB test, which was based on load-deformation characteristics of asphalt mixes. Also, analytical solutions and application of fracture mechanics in evaluating fracture properties of asphalt mixes that led to the development of a standard monotonic SCB test protocol were discussed. Overall, dynamic SCB test procedure is a good crack propagation assessment in the areas of asphalt mix fracture characterization.

2.5 Effect of N_{design} on Pavement Performance

An investigation of the effect of N_{design} values on performance of Superpave mixtures was made in North Carolina. Qarouach, et. al. (2015) investigated surface mixes in NC with nominal aggregate sizes of 9.5mm and 12.5mm with various traffic levels. Superpave design method was used to determine the asphalt content of each mixes. Asphalt pavement mixes were designed at N_{design} levels of 50, 75, 100, and 125. Asphalt Mixture Performance Tester (AMPT) device was utilized to measure optimum asphalt contents and dynamic modulus (E*). Then, E* data and binder properties were used as input in the AASHTO Darwin-ME software to predict rutting and fatigue performance of the asphalt mixtures. Then, relative performance recorded fatigue and rutting resistance for a specific mix was defined as the ratio of number of ESALs to failure for a given distress at a N_{design} level to the maximum ESALs (at 50 gyrations for fatigue and 125 gyrations for rutting). As they plotted the relative performance against asphalt content to determine optimum asphalt content, N_{design} was calculated as corresponding to the calculated optimum. Figure 11 illustrates the relative performance versus asphalt content. The number of gyrations for specific mixture is then determined from the plot of asphalt content vs gyrations as shown in figure 12.

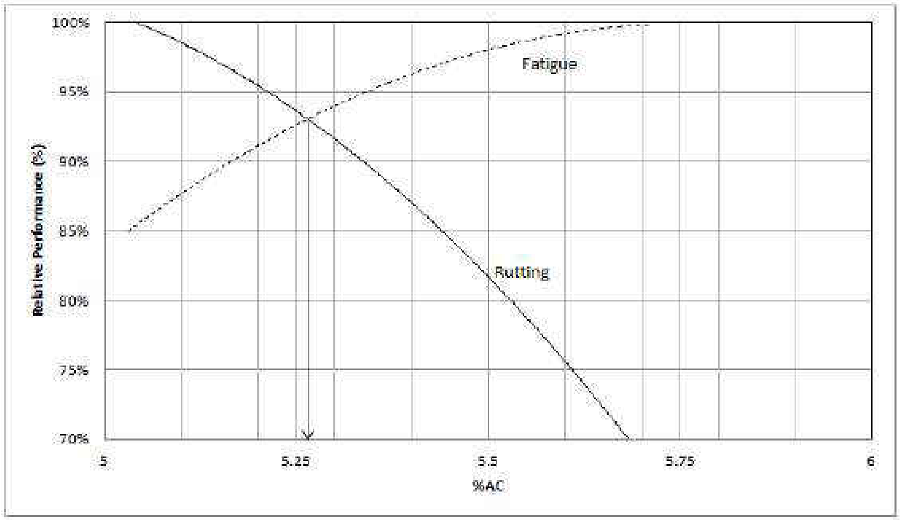


Figure 11. Relative Performance versus Asphalt Content (Qarouach, et.al. 2015)

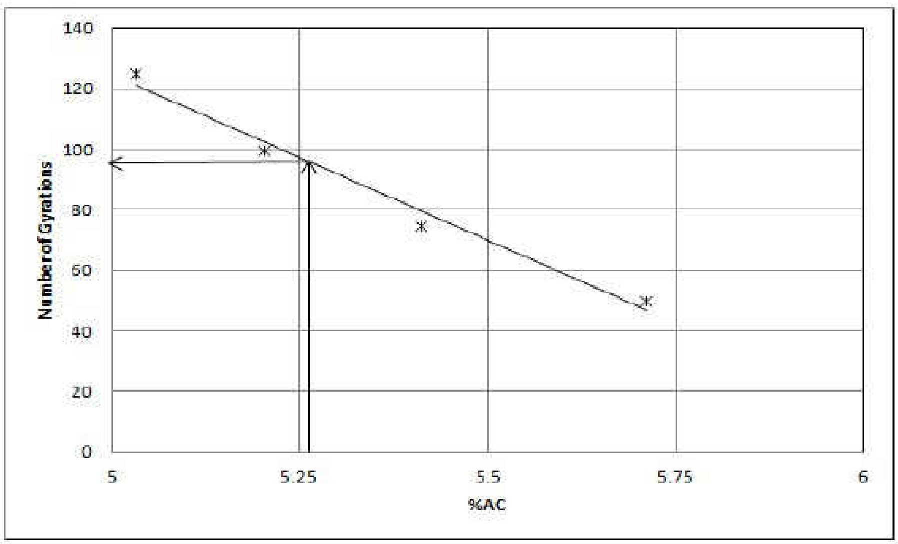


Figure 12. Number of Gyrations versus Asphalt Content (Qarouach, et.al. 2015)

Qarouach, et. al. (2015) observed that all the surface mixes with optimum asphalt content decreased with an increase in N_{design} level to which specimens were compacted. Also, mixtures stiffness is an extremely significant aspect of pavement design; it depends on the air void and asphalt contents of the mix and has effects on the fatigue performance of the pavement. Mixes with higher asphalt content binder exhibit lower fatigue cracking compared with lower asphalt content due to improved flexibility with excess asphalt binder. Furthermore, results from AMPT testing showed that the modulus of the mix at different temperature and frequencies increases with increase in N_{design} as observed from E* master curves for each mix at various N_{design} levels. This only prove the theoretical basis of the study that using a higher N_{design} for mix design requires lower binder content and results in a stiffer mix. Finally, they developed their final recommendation from two primary recommendations:

a. Effect of using a lower N_{design} on rutting and fatigue – improvement in pavement life with respect to fatigue life and corresponding increase in rutting (Qarouach, et. al. 2015).
b. Effect of using higher Ndesign - economic benefits from reduced use of asphalt binder in the mix weighed against the reduction in fatigue life (Qarouach, et. al. 2015).

2.6 Durability of Hot Mix Asphalt (HMA)

Durability of asphalt mixture is improved by additional asphalt binder content (Monismith, et. al. 1989). Additionally, it is enhanced by dense graded aggregate and uniformly compacted asphalt pavement. High asphalt content make asphalt protected against water because of its increased film thickness, and because of its increased average film thickness it decreases gap sizes between aggregates, thus making the mixture impenetrable to air and water (Monismith, et. al. 1989).

In 2015, Virginia Department of Transportation (VDOT) proposed changes to specifications of asphalt mix design. VDOTs proposal was to reduce design gyrations from 65 to 50 gyration. But, before modifications can be adopted Diefenderfer, et.al. (2018) performed a study to assess the effect the changes on mixture properties and lab performance. They evaluated eleven pairs of asphalt mix which consisted of typical specification of VDOT 65 gyration mix. Also, produced 50 gyration mix which was accorded to the proposed specification. They concluded that it had little effect on volumetric properties or gradations. Also, for the 50-gyration mixtures, core air voids were reduced, indicating the increase in durability of asphalt. Furthermore, because of increased asphalt binder, it resulted in an ability of the 50 gyration mixtures to be easily compacted in the field, which is expected to improve the durability of asphalt pavements in Virginia.

2.7 Effects of Asphalt Binder on Pavement Performance

2.7.1 Rutting

Rutting in asphalt pavement considered one of the major concerns in high temperature areas. Various research was made for rutting performance studies in asphalt mixes to mitigate the dilemma.

Moghaddam et. al. (2011) reviewed and highlighted previous research works conducted on the effects of using different types of additives and aggregate gradation on rutting resistance of asphalt mixtures. It was observed that mixtures higher asphalt content affected the rutting performance of asphalt pavement. Furthermore, rutting properties of asphalt can be improved by adding different additives such as polymers and fibers as mentioned in their paper. Fibers and polymers can absorb a certain amount of distresses imposed by repetitive loading and may help postpone deteriorations such as rutting in asphalt pavements.

Maupin, et. al. (2003) investigated various laboratory test samples of the field mixes (12.5mm and 9.5mm) to predicted changes in mix properties as extra asphalt was added. They performed Rutting Test in accordance with VTM-110, Virginia Test Method for Determining Rutting Susceptibility using the APA. It was concluded that additional asphalt did not increase rutting for some mixes and decreased slightly for some when 1.0% asphalt was added, which did not appear to be a problem. It was an indication that mixes did not contain enough asphalt to decrease shear strength and substantially increase rutting. Furthermore, most mixes improved as the asphalt content was increased.

2.6.2 Fatigue Cracking.

Various studies about fatigue cracking performance were also developed. Coleri et. al., (2018) characterized the cracking performance of asphalt pavements in Oregon by considering four (4) tests commonly used to evaluate fatigue cracking resistance. They proposed implementation of the most cost-effective and efficient test procedure for agencies and contractors. They concluded that SCB and IDT tests were the most practical and reliable tests that can be used to evaluate cracking resistance of asphalt mixtures. And that mixing method (laboratory or plant) does not have any significant effect on measured cracking performance. Binder content significantly affected the measured flexibility index (FI). A 0.7% increase in binder content raised the flexibility index by 2 to 3 times. They suggested that increasing binder content of asphalt mixtures currently used in the state can create significant savings and improve pavement longevity. Also, air-void content also significantly affected the measured FI. A 2% reduction in air-void content increases the flexibility index by 1.5 to 2 times. A higher flexibility index (FI) asphalt pavements may be more resistant to cracking and may increase longevity of asphalt pavements.

Maupin, et. al. (2003) additionally, investigated test samples for fatigue test included in the study from the previous section. Flexural beam fatigue test was performed in accordance with AASHTO Provisional Standard TP8-94, Standard Test Method for determining the fatigue life of compacted hot mix asphalt subjected to repeated loading. The results for 12.5mm and 9.5mm mixes which has 7.4, 6.6, and 7.5 percent of asphalt content with an additional 0, 0.5, and 1.0 percent, respectively. The target voids were lower than the 7.5 percent attained for the beams containing 1.0 percent additional asphalt; therefore, they believed that the fatigue life would have been slightly higher with lower target voids. As a result of the slight increase in fatigue life when asphalt content was increased, it indicated that the improvement of fatigue life is not extensive when 0.5 percent asphalt is added.

Moghaddam, et al. (2011) included fatigue resistance for asphalt mixtures with different types of additives and aggregate gradation. It was observed that mixtures with higher asphalt content showed lower fatigue life. They also recommended that fatigue properties of asphalt can be improved by adding different additives such as polymers and fibers.

2.6.3 Low Temperature Cracking.

As the city of Pittsburgh uses Superpave System for road pavement design consideration, Yeo, (2018) evaluated how to improve asphalt pavement in the city of Pittsburgh. It was mentioned in the study that thermal cracking and raveling increased as the asphalt aged. Pavement depth and the percentage of air voids in the pavement were important for the aging impacts of the asphalt pavement. Additionally, using the right performance grade (PG) of asphalt binder and aggregates were critical to prevent thermal cracking. It was recommended that in order to prevent thermal cracking in asphalt pavement, asphalt binder must be carefully selected, and it is crucial to study the right percentage of asphalt binder to be used in asphalt pavements in Pittsburgh.

A study on low temperature cracking was made by Li, et. al.(2007) in asphalt mixtures by using mechanical testing and acoustic emission methods. They investigated asphalt mixtures with the use of these methods to study microstructural phenomena and its corresponding effects on fracture behavior of asphalt mixtures at low temperatures. They tested eight asphalt mixtures, which presented a combination of factors such as aggregate type, asphalt content, and air voids with the use of SCB tests at three low temperatures. It was concluded that fracture resistance was dependent on temperature and significantly affected by type of aggregate and air void content. They did not see any significant effect on fracture resistance from asphalt content.

Li & Marasteanu (2010) evaluated low temperature fracture resistance for asphalt mixes with the use of SCB test. They evaluated six asphalt mixtures, which represented various factors such as binder type, binder modifier, aggregate type, and air voids. Three replicates were evaluated, and results indicated strong dependence of the low temperature fracture resistance on the test temperature. As the fracture energy was calculated from the experimental data, result showed that fracture resistance of asphalt mixtures was affected by type of aggregate and air void content. They reported that the low limit of the binder PG grade has significant effect on the fracture resistance of asphalt mixture at low temperature. The results show that mixture with high PG grade 58 binder has higher fracture energy than mixture with high PG grade 64 binder. Although, PG 64 mixture was discovered to have greater peak load than the PG 58 mixture. So, it is consistent with the expectation that PG 58-28 binder is known “softer” than the PG 64-28 binder (Li & Marasteanu, 2010). They concluded that the mixture with PG58 binder is more resistant to cracking than PG64 binder.

Marasteanu et. al., (2007) investigated low temperature cracking in asphalt pavement. They had two sets of materials that were evaluated using the current testing specification such as the creep and strength for asphalt binders and mixtures as well as newly developed protocols, such as the DCT test, single edge notched beam test and SCB test. Dilatometric measurements were performed on both asphalt binder and mixtures to determine the coefficient of thermal contraction. Discrete fracture and damage tools were utilized in their research to model crack initiation and propagation in pavement systems using the finite element method and TCMODEL. These were used with the experimental data from the field samples to predict performance and compare it to the field performance data. They concluded that asphalt binder properties represent a key factor in designing asphalt mixtures resistant to low temperature cracking. However, the current asphalt binder testing does not provide enough reliability to predict low temperature cracking of asphalt pavements. Furthermore, aggregate type has a significant effect on the fracture properties of similar types of mixtures made with the same asphalt binder. Also, low temperature cracking is influenced by volumetric properties like specific gravity of the mix (G_{sb}) or theoretical maximum specific gravity (G_{mm}). The study clearly established that the effect of temperature is significant as the behavior changes from brittle-ductile to brittle, therefore, when doing low temperature tests on asphalt mixtures, testing temperatures should be established relative to the expected low pavement temperature and/or relative to the low temperature Superpave Performance Grade (PG) for the location of interest.

Chapter 3

METHODOLOGY

3.1 General

Mixed aggregates that were used for the lab mix were collected from North Dakota Department of Transportation. Total of two (2) projects were selected in this research. Rutting, fatigue cracking and low-temperature cracking tests were done using Asphalt Pavement Analyzer (APA) Test, Semi-Circular Bending (SCB) Test, and Disc-Shaped Compact Tension (DCT) Test, respectively to develop the reduced N_{design} gyrations for the proposed project. The experimental plan for this research is summarized in Figure 13.

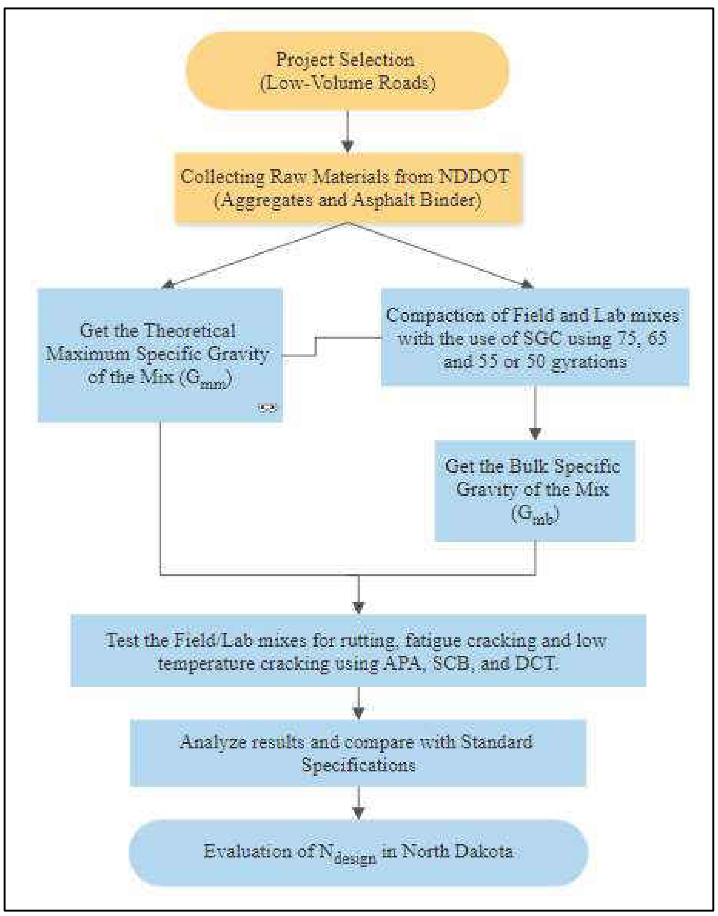


Figure 13. Experimental Plan

3.2 Project Selection

Two (2) projects of different aggregate sources and binder grades were selected from North Dakota Department of Transportation projects. Two projects were selected from a high volume highway class with Fine Aggregate Angularity (FAA) = 45 with binder grades 58H-28 and 58H-34, one project was selected from a medium volume highway class (FAA = 43) and binder grade 58S-28, and one project was from a low volume highway class (FAA = 40) with binder grade 58S-28. The gyratory compactive effort of the selected projects are 75 gyrations with an ESAL of 300,000. Table 3 below shows the project number, binder type and HMA Grade provided by NDDOT.

Table 3 Project Summary

Project Number	District	County	Binder Type	HMA Grade
NH-TRP-3-002(160)213	Devils Lake	Pierce	58H-28	FAA45
NH-3-281(127)125	Devils Lake	Eddy	58H-34	FAA45

3.3 Mix Preparations and Computations

3.3.1 Mass Determination of Aggregate and Asphalt Binder.

Particle size distribution of aggregate sample is critical to get the theoretical maximum specific gravity and bulk specific gravity of hot mix asphalt. To ensure the mix gradation of the given sample from NDDOT, it was decided to use the mechanical sifter to ensure that the distribution of particles was as close as in the field. There were four (4) projects in this research and we used the said mechanical sifter, as shown in Figure 14. Tables 4 and 5 show the distribution of batch weights and aggregate sizes based on

NDDOT Hot Mix Design Data. Equation 1 was used to calculate the amount of asphalt binder in grams.

$$W_{ac} = \frac{W}{\left(\frac{100 - AC}{100}\right)} - W \tag{1}$$

Where:

- W = Total weight of aggregates in grams
- W_{ac} = Total weight of asphalt binder in grams
- AC = Asphalt Binder in percent (%)

Figure 14. Common set-up of Mechanical Shaker

Table 4. Devils Lake, Eddy Project Batch Weight from NDDOT NH-3-281(127)125

Sieve Size	Batch Weights (g)	Batch Weights (%)
+3/8 Material	852.12	14.2
-3/8, +#4 Material	1,235.04	20.6
-#4 Material	3,912.84	65.2

Table 5. Devils Lake, Rugby Project Batch Weight from NDDOT NH-TRP-3-002(160)213

Sieve Size	Batch Weights (g)	Batch Weights (%)
+3/8 Material	828.36	13.8
-3/8, +#4 Material	1311.6	21.9
-#4 Material	3860.04	64.3

3.3.2 Theoretical Maximum Specific Gravity (G_{mm}).

The theoretical maximum specific gravity (G_{mm}) of HMA is an experiment to determine the specific gravity of HMA excluding the air voids. Hence, to obtain the G_{mm} of a hot mix asphalt, air voids must be eliminated, and the combination of aggregate and asphalt binder would be the theoretical maximum specific gravity. In this research "Rice" density test procedure was utilized to determine the theoretical maximum specific gravity. Figure 15 shows the asphalt mix getting ready for theoretical maximum specific gravity experiment at room temperature. A total of 750 grams of asphalt mix will be poured into the bowl together with water and the vacuum pump will be turned on and maintained at 25 mmHg for 15 minutes as seen in Figure 16. Figure 17 shows the usual set-up of the experiment.

Figure 15. Samples ready for Theoretical Maximum Specific Gravity (G_{mm}) testing

Figure 16. Set-up of bowl and Vacuum Gauge

Figure 17. Theoretical Maximum Specific Gravity (G_{mm}) Experiment Set-up

Due to the complexity of the value of G_{mm} which will be used to determine the air voids of compacted HMA, the standard procedure used was in accordance with AASHTO T209 to determine the theoretical maximum gravity of HMA. The typical values of theoretical maximum specific gravity of the mix ranges from 2.4 to 2.7 depending on the aggregate specific gravity and asphalt binder content. The theoretical maximum specific gravity of the mix was calculated from Equation 2.

$$Gmm = \frac{A}{(A + D - E)} \tag{2}$$

Where:

G_{mm} = Theoretical Maximum Specific Gravity of the mix

A = Sample mass in air (g)

D = Mass of bowl filled with water (g)

E = Mass of bowl and sample filled with water (g)

3.3.3 Mixing and Compaction of Asphalt Specimens.

Superpave gyratory compactor (SGC) was used to compact specimens at the desired compaction levels. In this research, 75, 65, 55 and 50 gyrations were used to determine the optimal asphalt content for a specified gyration. Following the AASHTO T312, lab mix aggregates were heated for 12 to 24 hours at a temperature of 325 °F and asphalt binder was heated at 290 °F for 3 to 4 hours. Also, mixing bowls, trays, asphalt spoons and wire whips were heated in the same oven as the asphalt binder at the same temperature (290 °F). When all the materials, aggregates and asphalt binder were ready, asphalt binder and aggregate were mixed using asphalt bowls and wire whisk as shown in Figure 18, and they were brought in an oven and short term aged for two (2) hours at a temperature of 280 °F. When the asphalt is ready after (2) two hours, mix will be inserted in the SGC machine as seen on Figure 19.

Figure 18. Mixing Bowls and Wire Whisks

Figure 19. Superpave Gyratory Compactor

While the hot mixed asphalt was heated in the oven, the molds, transfer pan, asphalt spoons were heated in a different oven at the same time. Additionally, the Superpave gyratory compactor was prepared and calibrated to 600 kPa.

After 2 hours of short-term aging of hot mix asphalt, the tray of asphalt mix was moved to a transfer pan and the mix was placed in the compaction mold. As the mold with asphalt mix was charged, the external angle was set to 1.25° ± 0.03° and with an internal angle of 1.16° ± 0.03°. After the desired compaction level (75, 65, 55 or 50) is achieved, the gyratory compactor automatically stops, and asphalt plugs are ready for determination of bulk specific gravity (G_{mb}) and percent air-voids.

3.3.4 Bulk Specific Gravity of the Mix (G_{mb}).

After the asphalt plugs were compacted, they were prepared for the determination of bulk specific gravity of the mix. For the preparation of this experiment, AASHTO T-166 was followed. Equation 3 below was used to determine G_{mb}.

$$Gmb = \frac{A}{B - C} \quad (3)$$

Where:

- G_{mb} = Bulk Specific Gravity of the mix
- A = mass of sample in air (g)
- B = mass of SSD sample in air (g)
- C = mass of sample in water (g)

There are different procedures under AASHTO T 166. In this research, saturated surface dry (SSD) was the method used. The SSD is the most common method that calculates the specimen volume by subtracting the mass of the specimen under water from the mass of an SSD specimen. To get the following parameters, mass of sample in air was determined with a calibrated scale. After recording the mass of sample in air, the asphalt plug was submerged in water with a temperature of 25°C (77°F) for 4 minutes as seen in Figure 20. Then, after recording the mass of sample in water, the asphalt plug must be quickly dried with a damp towel and the surface dry mass of the sample would be recorded. The typical values of bulk specific gravity of mixture ranges from 2.2 to 2.5 depending upon the bulk specific gravity of the aggregate, the asphalt binder content, and amount of compaction. Figure 21 shows the prepared compacted asphalt mix samples for G_{mb} testing.

Figure 20. Sample Asphalt plug weighed in air and Sample submerged under water

Figure 21. Asphalt Samples ready for G_{sb} Experiment

3.3.5 Percent (%) Air Voids of the Specimen (Va).

Once Theoretical Maximum Specific Gravity (G_{mm}) and Bulk Specific Gravity of the Mix (G_{mb}) are known, percent air voids can be calculated. It is calculated by comparing G_{mb} and G_{mm} and quantified as a percentage. Through the determination of air voids, it is assumed that the difference of both values is due to air. The computation to get the percent air voids is calculated with Equation 4.

$$AV = \left(\frac{Gmm - Gmb}{Gmm}\right) \times 100\% \qquad (4)$$

Where:

- AV = Air Voids (%)
- G_{mm} = Theoretical Maximum Specific Gravity of the Mix
- G_{mb} = Bulk Specific Gravity of the Mix

3.4 Performance Testing

Rutting, low temperature cracking and fatigue cracking were determined using Asphalt Pavement Analyzer (APA), Semi-Circular Bend (SCB) Test and Disk-shaped Compaction Tension (DCT) Test, respectively. All the asphalt plugs must meet the 7.0±0.5% air void content criteria to mimic constructed asphalt pavements. Figure 22 shows the machines used to prepare sample specimens.

Figure 22. Cutting, Drilling and Sawing Machines

3.4.1 APA Test.

Asphalt Pavement Analyzer (APA) was utilized to determine the rutting performance following AASHTO T340. The APA is a temperature-controlled wheel tracking device. The machine measures the rutting that develops from laboratory compacted specimens. The APA features controllable wheel load and contact pressure that represents actual field conditions. The SGC was used to compact the cylindrical specimens that are 6 inches (150mm) in diameter and 3 inches (75mm) in height that was in accordance with AASHTO T 340. Rutting in the asphalt specimens was induced with the use of a pneumatic hose loaded oscillating aluminum wheel, the hose is inflated to 100 psi and placed over the compacted asphalt specimens. Samples were conditioned 24-hours submerged in a 58°C water bath followed by APA testing, and after 8,000 cycles rut depths is recorded. In analyzing the results, APA performance specification is based on the evaluation of rutting performance mixes from North Dakota mixes that has an average of 7 mm rut depth (Suleiman, 2005).

3.4.2 DCT Test.

Disk-Shaped Compact Tension (DCT) test was utilized to determine the fracture energy of lab compacted specimens following ASTM D7313. The DCT is used as performance-type test specification to control different forms of cracking, such as thermal, reflective, and block cracking of pavements surfaced with asphalt pavement. Sample specimens were conditioned for 8 hours at low temperature PG+10°C of the binder. During the test, a constant Crack Mouth Opening Displacement (CMOD) rate of 0.017 mm/s was maintained.

Disk Shaped specimen is pulled apart until the post peak level has generated to 0.02 lb (0.1 kN). As for the geometry of the specimen, it has a 6-in (150mm) diameter, 2-in (50-mm) thick overall dimension with two 1-in (25mm) holes on either side of a 2.46-in (62.5-mm) notch cut into a flattened portion of the circumference as shown in Figure 23. Figure 24 shows the typical set-up of DCT test.

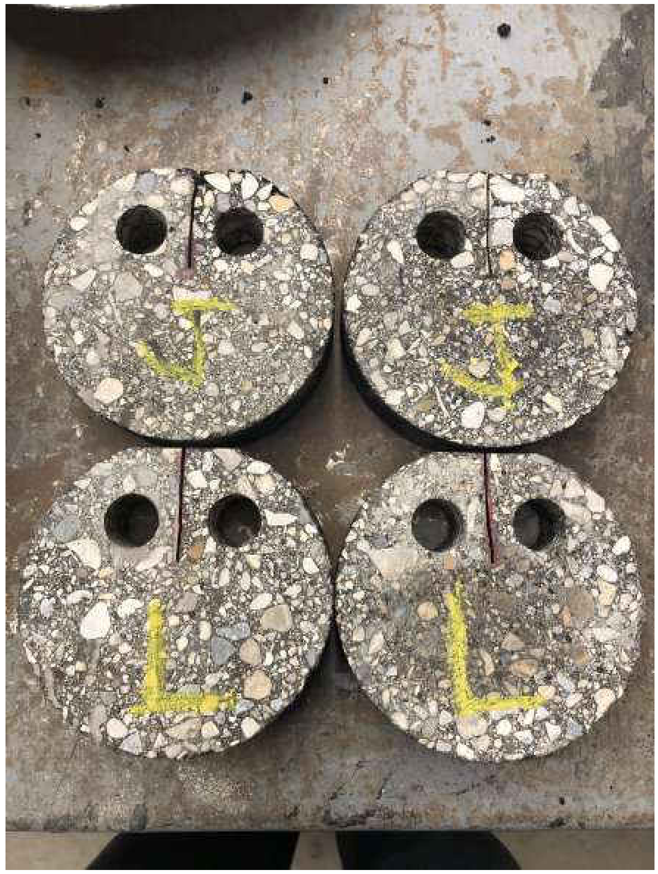


Figure 23. DCT Samples ready for Testing

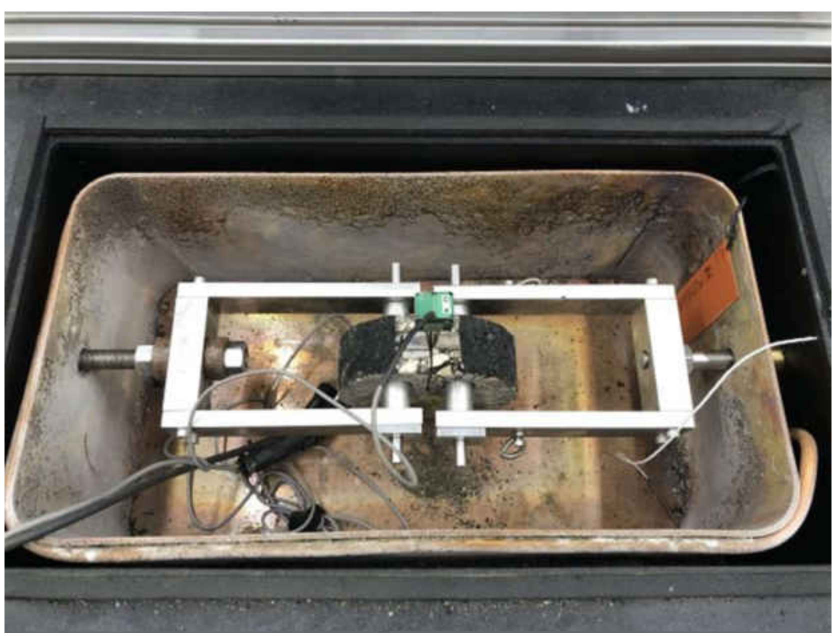


Figure 24. DCT Sample set-up

Fracture energy (*Gf*) is calculated by determining the area under the load, CMOD curve normalized the initial ligament length and thickness. The larger the *Gf*, the better the cracking resistance of the asphalt mixture is. The typical coefficient of variation (COV) for the DCT test for virgin mixtures is around 10 percent. Figure 25 shows a sample graph of CMOD versus Load (kN).

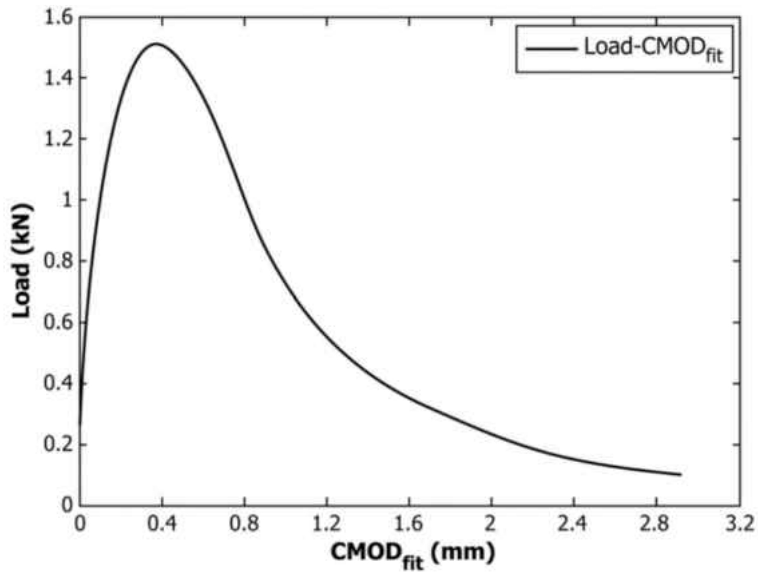


Figure 25. Typical Load vs CMOD$_{fit}$

3.4.3 SCB Test.

Fatigue resistance was determined in accordance with AASHTO TP124-16. Illinois-Flexibility Index Tester (IFIT) protocol was used for samples with sizes of 50±2mm and were tested using the SCB to determine fatigue cracking resistance of laboratory compacted samples. The samples were conditioned for 2 hours and tested at 25°C. Fracture energy is the total area under load vs displacement curve and FI is the slope of the curve post peak load. FI was calculated using Equation 5. Figure 26 shows typical asphalt specimens ready for testing. Figure 27 shows the typical set-up for an SCB test. Typical load vs displacement is shown in figure 28.

$$FI = \frac{Gf}{|m|}\, x\, A \qquad (5)$$

Where:

FI = Flexibility Index

Gf = Fracture Energy (J/m^2)

$|m|$ = Absolute value of post – peak load slope (kN/mm)

A = conversion factor = 0.01

Figure 26. SCB Samples ready for Testing

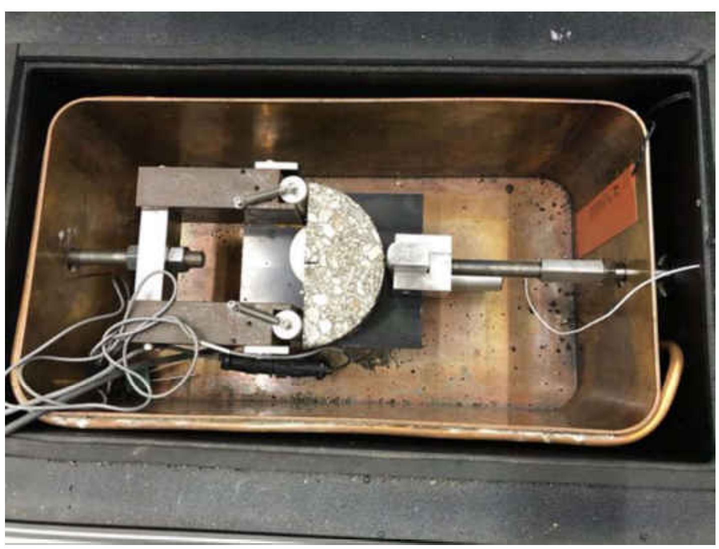


Figure 27. SCB Sample Set-Up

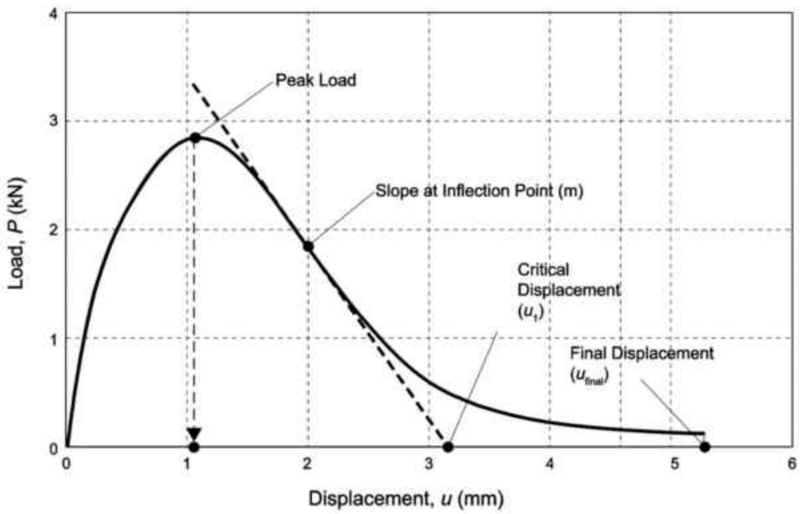


Figure 28. Typical Load Vs Displacement

Chapter 4

RESULTS AND DISCUSSIONS

4.1 Theoretical Maximum Specific Gravity

Table 6 shows the summary of theoretical maximum specific gravity provided by NDDOT and laboratory mixes made in UND Civil Engineering Lab. The G_{mm} using laboratory mix was slightly different from NDDOT because of various issues like aggregate gradation, computation of asphalt content, environment in the laboratory and workmanship.

Table 6. Theoretical Maximum Specific Gravity Comparison between NDDOT and Lab Mixes

Project Number	AC Binder	AC %	G_{mm} Values	
			NDDOT	UND Lab Mix
Pierce County, Devils Lake - US 2, 1 MI E of RUGBY, EB/WB				
NH-TRP-3-002(160)213	58H -28	5.0	2.500	2.517
	58H -28	5.5	2.481	2.487
	58H -28	6.0	2.466	2.459
	58H -28	6.5	2.451	2.436
	58H -28	7.0	-	2.420
	58H -28	5.9	-	2.456
Eddy County, Devils Lake – NEW ROCKFORD, ND				
NH-3-281(127)125	58H – 34	4.5	2.514	-
	58H – 34	5.0	2.492	2.500
	58H – 34	5.5	2.472	2.482
	58H – 34	6.0	2.456	2.467
	58H – 34	6.5	-	2.459
	58H – 34	7.0	-	2.444
	58H – 34	5.2	2.489	2.487

4.2 Determination of Optimum Asphalt Content

Optimum asphalt content was determined by accomplishing relating air voids and binder content linear graph. The best fit line was utilized to determine the required asphalt content at 4% air voids. The linear equation in each graph was used to get the asphalt content at 4% air voids which is required by NDDOT. Gyrations 75, 65 and 55 was applied to SGC to determine the optimum asphalt content on their respective compaction effort. Figures 29, 30, and 31 show the linear relationship between air voids and binder content for Devils Lake project located in Pierce County (Rugby Project).

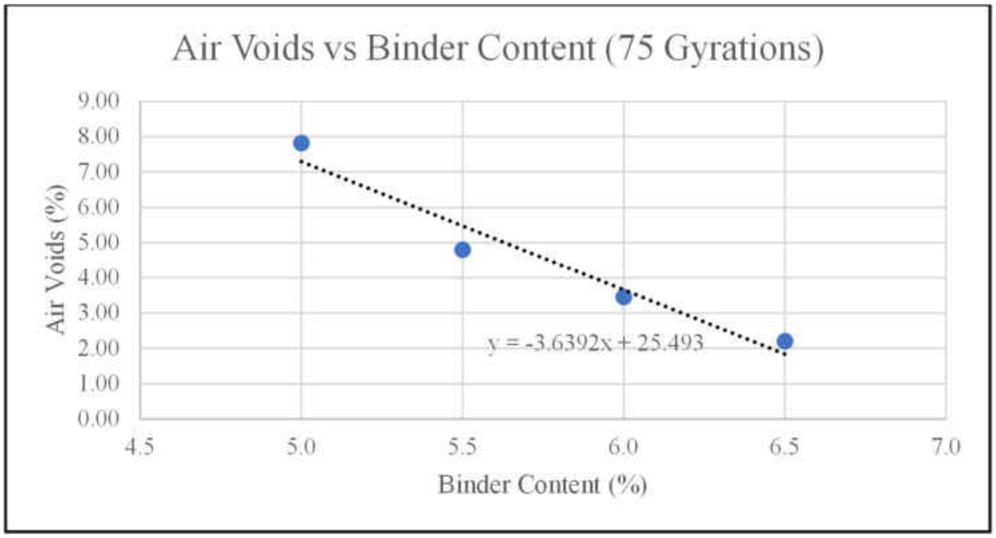


Figure 29. Air Voids vs Binder Content (75 Gyrations) – Rugby Project

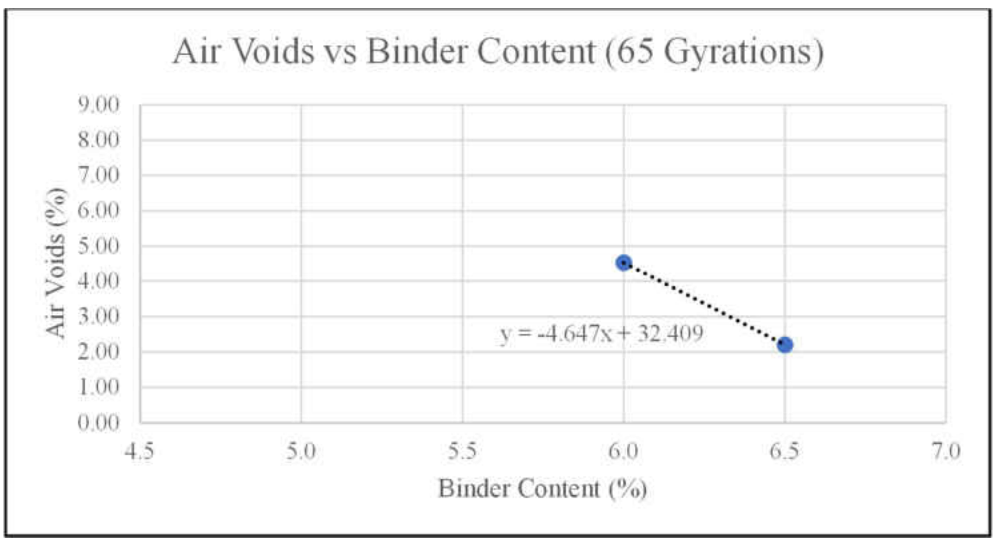


Figure 30. Air Voids vs Binder Content (65 Gyrations) – Rugby Project

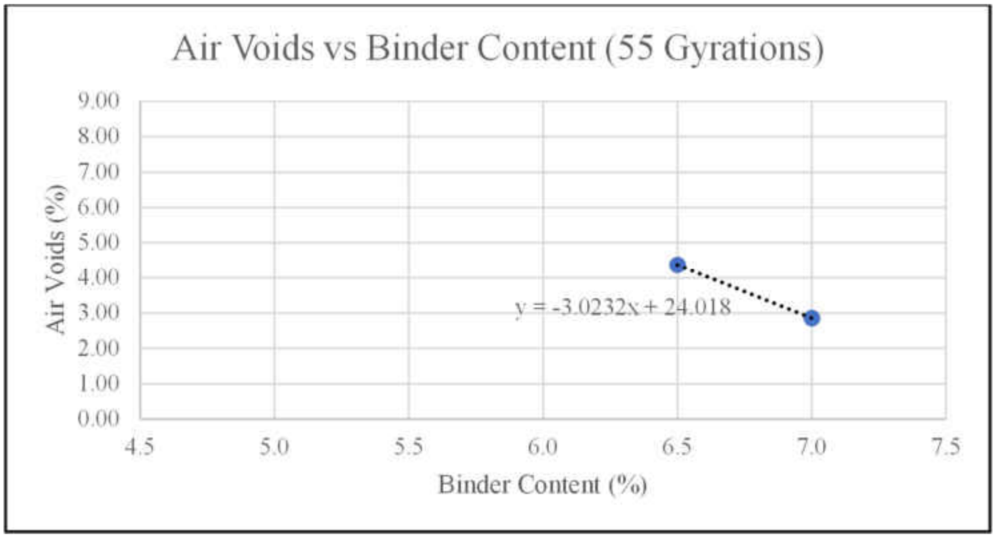


Figure 31. Air Voids vs Binder Content (55 Gyrations) – Rugby Project

The linear relationship for 75, 65 and 55 gyrations for Rugby Project was determined to have an asphalt content of 5.9%, 6.1%, and 6.6% respectively at 4% air voids. Linear relationship in figures 32, 33 and 34 was used to determine the optimum asphalt content for Eddy Project. Asphalt content at 4% air voids was determined to have 5.6%, 6.1% and 6.6% for 75, 65 and 55 gyration, respectively.

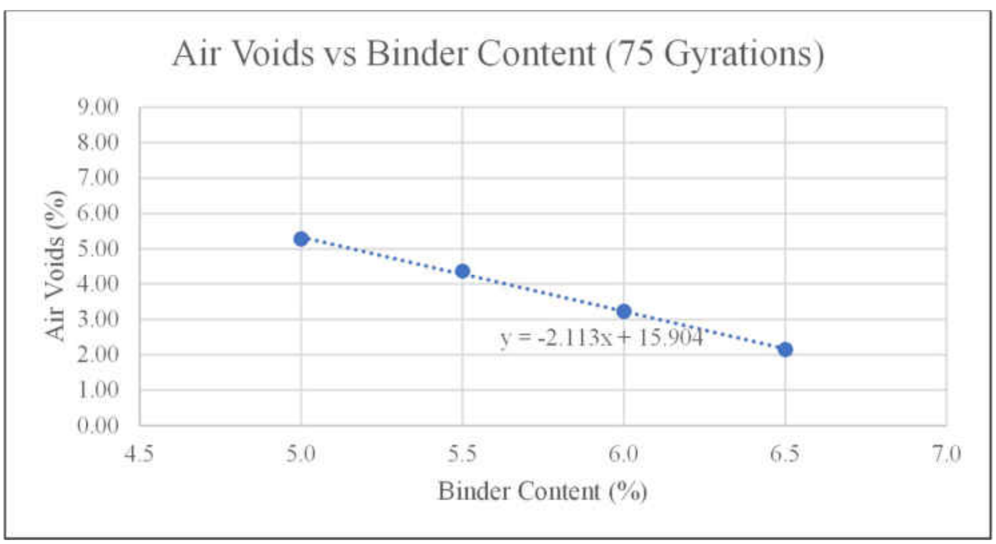


Figure 32. Air Voids vs Binder Content (75 Gyrations) – Eddy Project

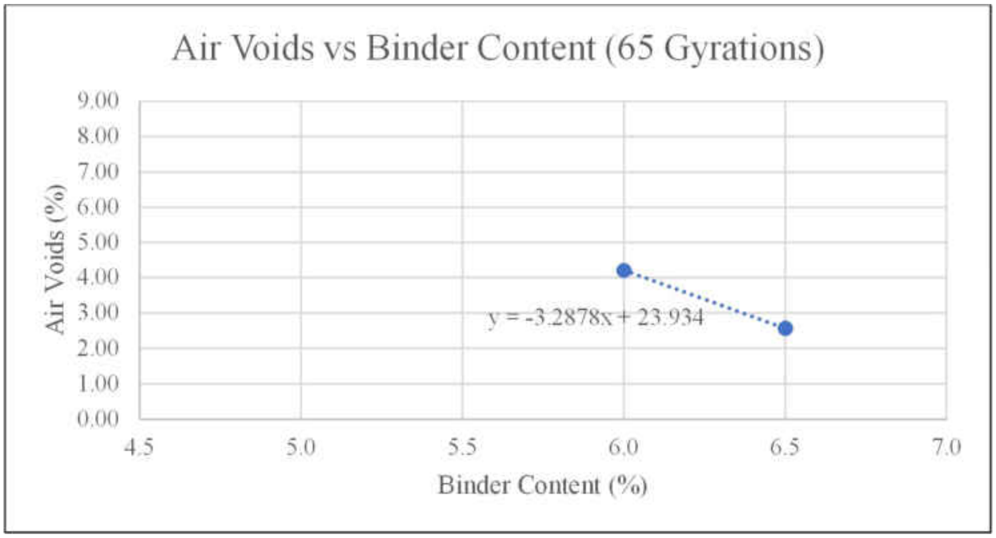


Figure 33. Air Voids vs Binder Content (65 Gyrations) – Eddy Project

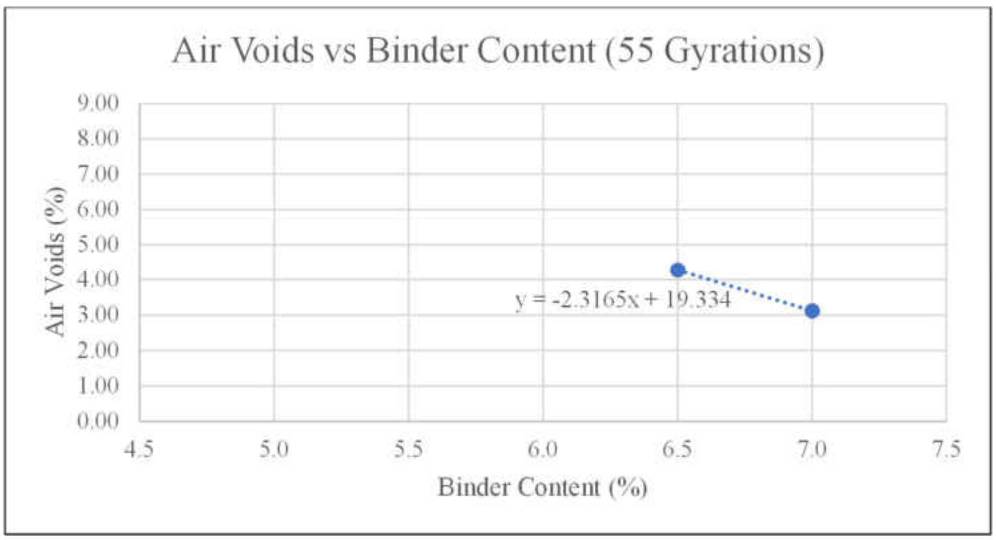


Figure 34. Air Voids vs Binder Content (55 Gyrations) – Eddy Project

4.2 Rutting Performance

Due to unavoidable circumstances the research was not able to perform Asphalt Pavement Analyzer (APA) test. It will be done in the future to validate and check the rutting depths for the Rugby and Eddy projects.

4.3 Low Temperature Cracking Performance

The results of low temperature tests are shown in tables 7 and 8. Asphalt specimens were compacted at air voids of 7±0.5% to simulate the field conditions.

Fracture energy in the ranges of 350 – 400 J/m^2 is considered borderline, and permissible on less critical projects (Newcomb, 2018). Results indicate that most of the mixes satisfied the minimum fracture energy of 400 J/m^2 except from the field mix with 5.2% asphalt content from Eddy Project had the lowest fracture energy of 214 J/m^2 as seen in Table 7. The highest recorded fracture energy is 735 J/m^2 seen at 65 gyrations with specimen ID A1 in Rugby project with asphalt binder of 58H-34 and 6.1% asphalt content.

Table 7. Eddy Project (PG 58H-34) DCT Results

Specimen ID	Fracture Energy (J/m^2)	Status
75 Gyration	5.6% Asphalt Content	
B1	427.00	Pass
B2	452.00	Pass
C1	414.00	Pass
C2	488.00	Pass
65 Gyration	6.1% Asphalt Content	
B1	441.00	Pass
B2	477.00	Pass
C1	556.00	Pass
C2	502.00	Pass
55 Gyration	6.6% Asphalt Content	
B1	510.00	Pass
B2	434.00	Pass
C1	541.00	Pass
C2	510.00	Pass
Field Mix 75 G's	5.2% Asphalt Content	
B1	295.00	Fail
B2	214.00	Fail
C1	511.00	Pass
C2	265.00	Fail

Table 8. Rugby Project (PG 58H-28) DCT Results

Specimen ID	Fracture Energy (J/m^2)	Status
75 Gyration	5.9% Asphalt Content	
J1	569.00	Pass
J2	556.00	Pass
L1	543.00	Pass
L2	578.00	Pass
65 Gyration	6.1% Asphalt Content	
A1	735.00	Pass
A2	658.00	Pass
B1	604.00	Pass
B2	490.00	Pass
55 Gyration	6.6% Asphalt Content	
A1	620.00	Pass
A2	590.00	Pass
B1	568.00	Pass
B2	570.00	Pass
Field Mix 75 G's	5.5% Asphalt Content	
A1	422.00	Pass
A2	406.00	Pass
C1	451.00	Pass
C2	432.00	Pass

Results show that the max low temperature performance for Eddy project with asphalt binder of 58H-34 is at 55 gyrations with an average of 498.75 J/m^2 as seen in

Table 9. Rugby project at 65 gyrations with asphalt binder of 58H-28 has an average of 621.75 J/m^2 as shown in Table 10.

Table 9. Summary of DCT Results for Eddy Project (PG 58H-34)

Low temperature Performance - Eddy Project				
	AC Binder	Average (J/m^2)	SD (J/m^2)	COV (%)
75 Gyration	58H-34	445.25	32.57	7.3%
65 Gyration	58H-34	494.00	48.33	9.8%
55 Gyration	58H-34	498.75	45.57	9.1%
Field Mix - 75 Gyrations	58H-34	321.25	130.84	40.7%

Table 10. Summary of DCT Results for Rugby Project (PG 58H-28)

Low temperature Performance - Rugby Project				
	AC Binder	Average (J/m^2)	SD (J/m^2)	COV (%)
75 Gyration	58H-28	561.50	15.29	2.7%
65 Gyration	58H-28	621.75	102.98	16.6%
55 Gyration	58H-28	587.00	24.14	4.1%
Field Mix - 75 Gyrations	58H-28	427.75	18.84	4.4%

Figures 35 and 36 show the comparison between different gyrations with fracture energies. Field mixes with 75 gyrations that were provided by NDDOT with asphalt contents of 5.2% for Eddy project and 5.5% for Rugby project, it was evident that they had the least low temperature cracking performance. Both field mixes, Eddy and Rugby, had the lowest cracking resistance with a value of 321.25 J/m^2 and 427.75 J/m^2, respectively. Field mix from Eddy project did not reach the minimum cracking resistance of 400 J/m^2. Figure 37 shows the typical specimens after DCT test.

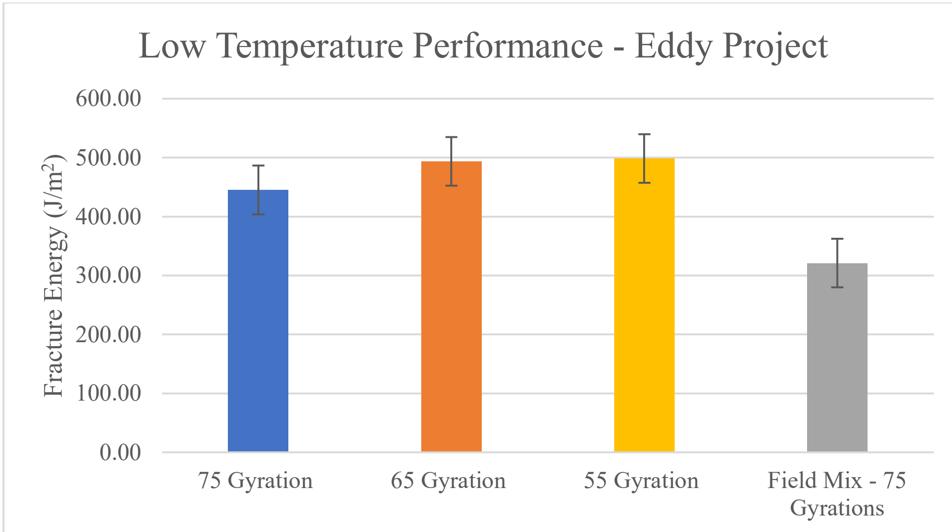


Figure 35. Eddy Project PG 58-34 Low Temperature Performance

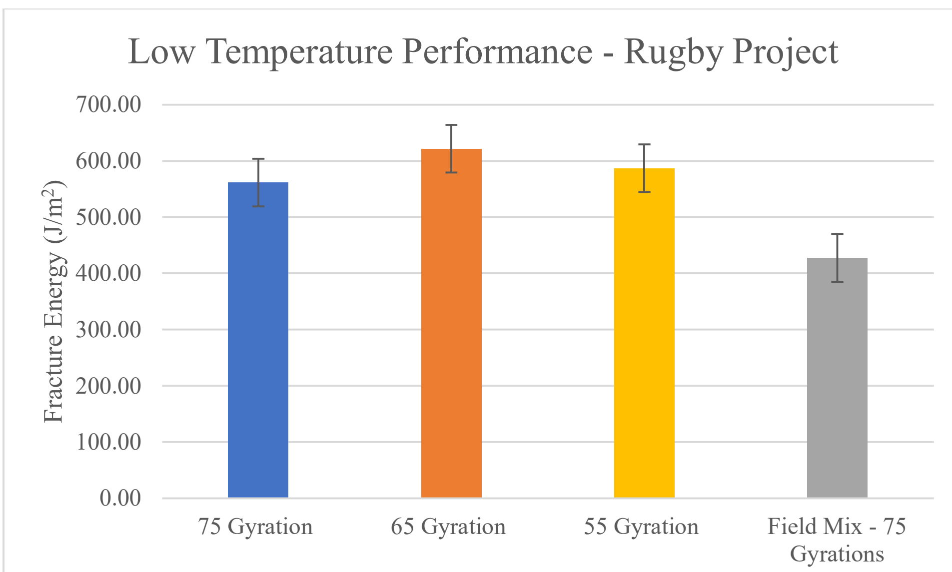


Figure 36. Rugby Project PG 58-28 Low Temperature Performance

Figure 37. Typical sample after DCT test

4.4 Fatigue Cracking Performance

The results of fatigue tests are shown in Table 11 and 12. Like previous performance test, specimens were compacted at 7±0.5% to mimic the field conditions.

Table 11. Eddy Project SCB Results

Specimen ID	Fracture Energy (J/m2)	Flexibility Index (FI)
75 Gyration	5.6% Asphalt Content	
A1	1951.04	13.55
A2	1905.36	15.12
A3	1288.62	13.42
A4	1947.05	14.86
65 Gyration	6.1% Asphalt Content	
A1	2238.19	22.16
A2	1743.55	17.97
A3	2149.66	19.9
A4	1937.86	17.46
55 Gyration	6.6% Asphalt Content	
A1	2171.81	16.97
A2	2380.4	24.04
A3	1777.31	12.97
A4	1459.05	11.14
Field Mix 75 G's	5.2% Asphalt Content	
A1	1112.46	3.53
A2	1254.51	3.15
A3	1579.83	5.6
A4	1317.12	5.33

Table 12. Rugby Project SCB Results

Specimen ID	Fracture Energy (J/m2)	Flexibility Index (FI)
75 Gyration	5.9% Asphalt Content	
K1	2567.46	16.78
K2	2611.09	10.70
K3	3651.82	20.99
K4	2838.55	15.02
65 Gyration	6.1% Asphalt Content	
D1	2182.37	13.47
D2	3230.56	11.26
D3	2676.4	11.64
D4	2623.01	13.59
55 Gyration	6.6% Asphalt Content	
D1	2903.32	23.99
D2	2601.17	16.16
D3	3079.43	16.04
D4	3099.72	20.00
Field Mix 75 G's	5.5% Asphalt Content	
B1	2066.44	8.07
B2	2422.56	6.46
B3	2311.19	6.72
B4	2491.19	6.45

Results shows that the highest average fracture energies for Eddy and Rugby projects are 2017.32 J/m^2 at 65 gyrations and 2920.91 J/m^2 at 55 gyrations respectively as seen in Tables 13 and 14.

Table 13. Summary of SCB Test Results for Eddy Project

Fatigue Cracking Performance - Eddy Project				
	AC Binder	Average (J/m^2)	SD (J/m^2)	COV (%)
75 Gyration	58H-34	1773.02	323.59	18.3%
65 Gyration	58H-34	2017.32	221.78	11.0%
55 Gyration	58H-34	1947.14	410.39	21.1%
Field Mix - 75 Gyrations	58H-34	1315.98	195.63	14.9%

Table 14. Summary of SCB Test Results for Rugby Project

Fatigue Cracking Performance - Rugby Project				
	AC Binder	Average (J/m^2)	SD (J/m^2)	COV (%)
75 Gyration	58H-28	2917.23	503.94	17.3%
65 Gyration	58H-28	2678.09	429.73	16.0%
55 Gyration	58H-28	2920.91	230.68	7.9%
Field Mix - 75 Gyrations	58H-28	2322.85	186.34	8.0%

Table 15 and 16 shows the flexibility index and the highest recorded average for Eddy and Rugby projects is 19.37 and 19.05, respectively. The FI values show that both projects are more than 10 generally provide excellent cracking resistance. For field mixes, Eddy project got an average of 4.40 which shows poor cracking resistance.

Table 15. Flexibility Index – Eddy Project

Flexibility Index - Eddy Project				
	AC Binder	Average	SD	COV (%)
75 Gyration	58H-34	14.24	0.88	6.2%
65 Gyration	58H-34	19.37	2.13	11.0%
55 Gyration	58H-34	16.28	5.72	35.1%
Field Mix - 75 Gyrations	58H-34	4.40	1.24	28.2%

Table 16. Flexibility Index - Rugby Project

Flexibility Index - Rugby Project				
	AC Binder	Average	SD	COV (%)
75 Gyration	58H-28	15.87	4.26	26.9%
65 Gyration	58H-28	12.49	1.21	9.7%
55 Gyration	58H-28	19.05	3.77	19.8%
Field Mix - 75 Gyrations	58H-28	6.93	0.77	11.2%

Figures 38 and 39 show comparison of performance between different gyrations. The bar graphs show that the highest cracking resistance is at 65 gyrations for Eddy and 55 gyrations for Rugby which suggests that asphalt mixes that have higher asphalt content have higher fracture energies.

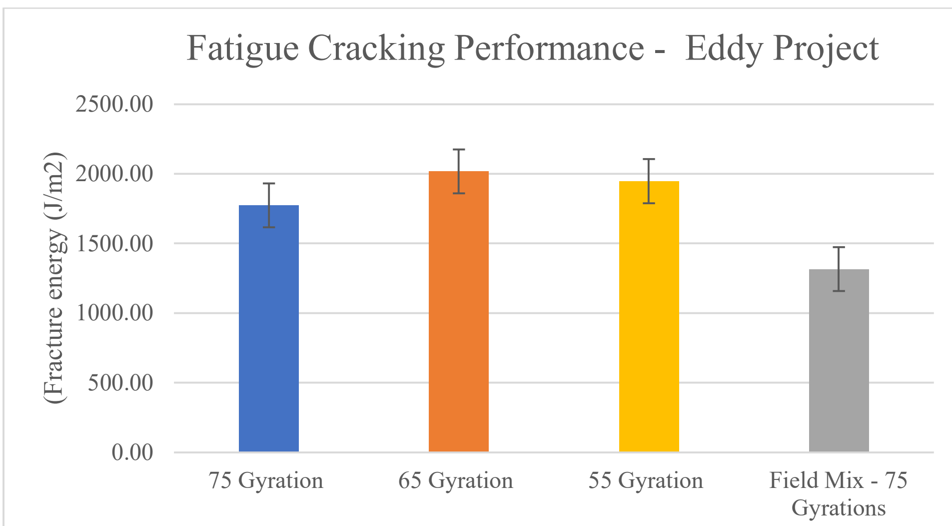


Figure 38. Eddy Project Fatigue Cracking Performance

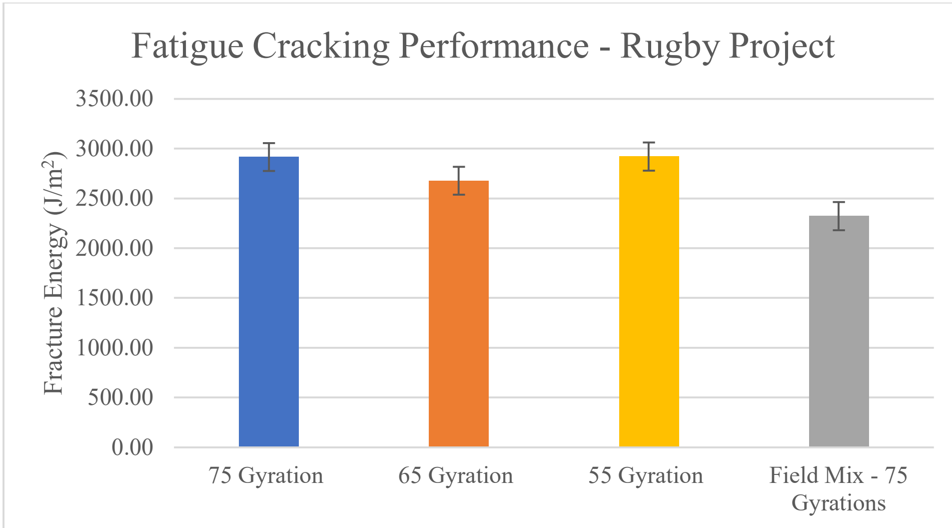


Figure 39. Rugby Project Fatigue Cracking Performance

Figures 40 and 41 confirms that lower gyrations with higher asphalt content provided excellent fatigue cracking resistance. Lab mixes show better flexibility index compared to field mixes. Figure 42 shows the typical sample specimen after SCB test.

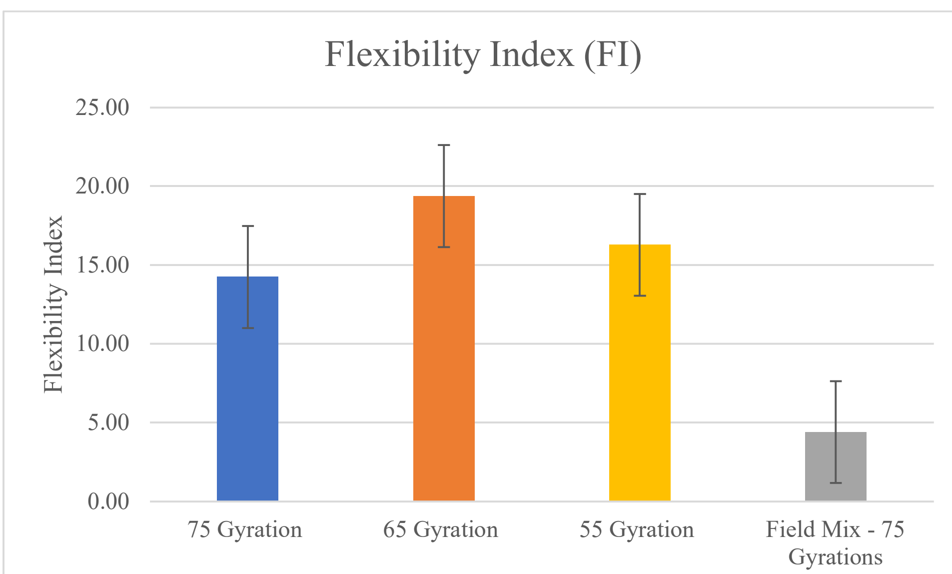


Figure 40. Flexibility Index – Eddy Project

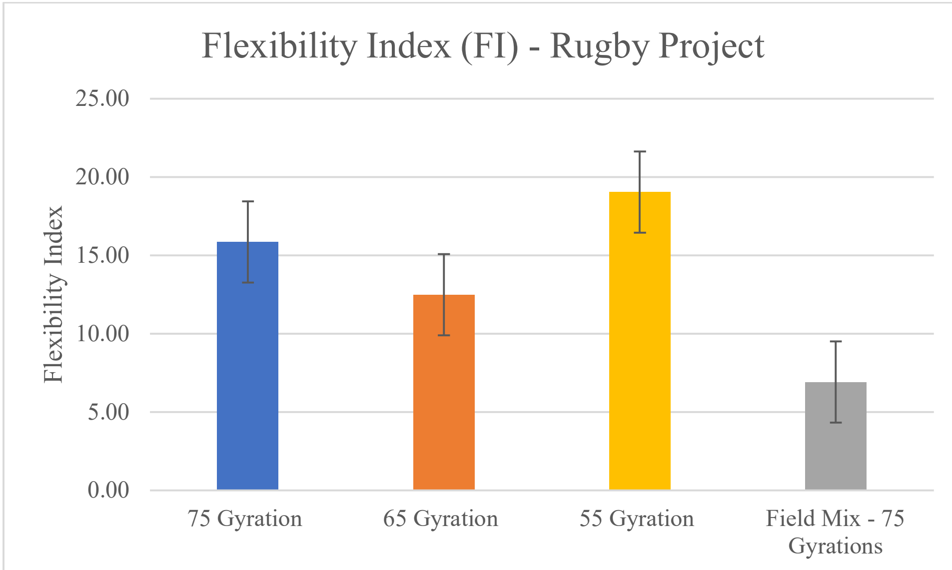
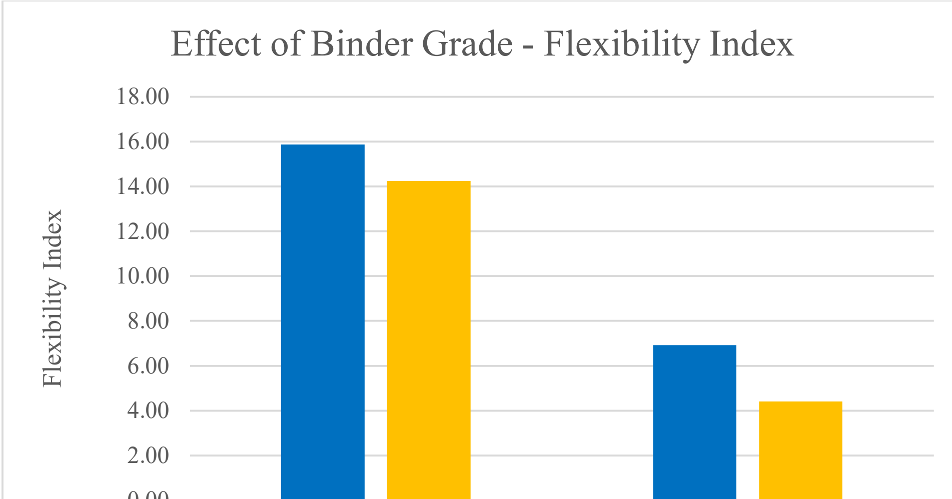


Figure 41. Flexibility Index – Rugby Project

Figure 42. Typical sample after SCB test

Figure 43 show the effect of binder grade type from both Rugby and Eddy project. Lab mixes has better FI value compared to field mixes. It should be noted that variations could be due to aggregate gradation, environment, workmanship, and binder content. PG 58H-28 has good performing FI value at 15.87 than PG 58H-34 which had an FI value of 14.24. For field mixes PG 58-28 had an intermediate performing FI value compared to PG 58-34 with FI values at 6.93 and 4.40, respectively.



	75 Gyrations	Field Mix - 75 Gyrations
PG 58H-28	15.87	6.93
PG 58H-34	14.24	4.40

Figure 43. Effect of Binder Grade comparison

Chapter 5

CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusions

The following conclusions are made based on the results from Eddy and Rugby projects. Due to unforeseen lab circumstances APA tests for rutting were not conducted.

- Fatigue and Low Temperature Cracking were done to Rugby and Eddy projects and showed good performing asphalt pavements.
- Gyration number N_{design} = 75 is possibly too high for High and Low volume roads as the fracture energies shows good result with lower gyration numbers for both high volume roads (FAA 45) projects, but, APA test must be evaluated to get the recommended balanced mix design.
- An increase in asphalt content will increase its cracking resistance but will reduce its rutting resistance of asphalt. In contrast, decreasing the asphalt content will increase its rutting resistance but will reduce its cracking resistance.
- PG 58-28 has better fracture energy compared to PG 58-34.

Low Temperature Cracking Performance (DCT Test)

1. Eddy Project
   - 81.25% of the lab mix fulfilled the minimum fracture energy of 400 J/m^2.
   - The fracture energy ranges from 414 J/m^2 to 556 J/m^2. Field mix that failed had the lowest fracture energy of 214 J/m^2.
   - The highest average fracture energy recorded was at 498.75 J/m^2 with asphalt content of 6.6% at 55 gyrations.
   - Lab mix has higher fracture energy than field mixes.

2. Rugby Project
    - 100% of the lab mixes fulfilled the minimum fracture energy of 400 J/m2.
    - The fracture energy ranges from 406 J/m2 to 735 J/m2.
    - The highest average fracture energy recorded was at 621.75 J/m2 with asphalt content of 6.1% at 65 gyrations.
    - Lab mix has higher fracture energy than field mixes.

Fatigue Cracking Performance (SCB Tests)

1. Eddy Project
    - The maximum average fracture energy is recorded at 2017.32 J/m2 at 65 gyrations with 6.1% asphalt content. The lowest recorded was for the field mix which was 1315.98 J/m2 at 5.2% asphalt content.
    - FI ranged from 4.4 to 19.37. Field mix has the lowest FI of 3.15 which indicates that it has intermediate fatigue cracking performance.
    - FI values of 2.0 and 6.0 are the cut-off values distinguishing poor – (less than 2.0), intermediate – (2.0 to 6.0), and good performing (greater than 6.0) (Al-Qadi, et al., 2015).
2. Rugby Project
    - The maximum average fracture energy was 2920.91 J/m2 at 55 gyrations with 6.6% asphalt content. The lowest value was 2320.91 J/m2 for the field mix at 5.5% asphalt content.
    - FI ranges from 6.93 to 19.05. Field mix has the lowest FI of 6.93 which indicates that it has good fatigue cracking performance.

- FI values of 2.0 and 6.0 are the cut-off values distinguishing poor – (less than 2.0), intermediate – (2.0 to 6.0), and good performing (greater than 6.0) (Al-Qadi, et al., 2015).

5.2 Recommendations

- NDDOT should explore ways to integrate new specifications for additional asphalt binders in asphalt mixes.
- Asphalt Pavement Analyzer (APA) test must be evaluated in the future to validate and to recommend a design number of gyrations that will help to balance the mix design in both rutting and cracking in North Dakota.
- Experiments for FAA 43 and FAA 40 projects must be done in the future to get the rutting, fatigue and low temperature performances for medium and low volume roads.

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Appendix

Table 17. Eddy Project Specimen Samples

Specimen ID	Unc. Gmm	Height (mm)	V-Press	Angle	Dry (g)	SSD (g)	Wet(g)	Gmb
Project: NH-3-281(127)125: Devils Lake, Eddy								
75 Gyrations								
5.2A_FM	95.0%	112.87	603.43	1.22	4694.8	4711	2745	2.388
5.2B_FM	94.7%	113.13	601.94	1.23	4709.2	4728.6	2760.6	2.3929
5.0A	95.2%	117.04	602.93	1.29	4839.8	4854.1	2813.8	2.3721
5.0B	93.9%	118.58	602.98	1.27	4896.1	4910.2	2839.5	2.3645
5.0E	94.8%	117.5	602.44	1.23	4860.2	4872.1	2825	2.3742
5.5A	94.4%	119	600.2	1.25	4892.5	4905	2832.6	2.3608
5.5B	93.8%	119.77	600.95	1.23	4908.2	4922	2836.3	2.3533
5.5C	94.2%	119.25	601.44	1.2	4857.5	4870	2792.5	2.3381
5.5D	95.8%	117.3	599.71	1.27	4874.6	4883	2834.2	2.3792
5.5E	95.3%	117.92	602.44	1.23	4877.9	4889.7	2830.2	2.3685
6.0A	97.8%	116.11	599.21	1.27	4904.7	4906.2	2873.5	2.4129
6.0B	97.8%	116.22	599.71	1.26	4907.4	4909.8	2880.3	2.418
6.0C	95.6%	118.79	602.68	1.21	4931.2	4938.5	2864.5	2.3776
6.0D	96.3%	117.79	602.79	1.27	4926.2	4932.1	2870.8	2.3899
6.0E	96.5%	117.76	600.95	1.23	4907.4	4913.8	2856.9	2.3858
6.5A	99.2%	115.96	600.7	1.3	4906.8	4907.8	2877.6	2.4169
6.5B	98.1%	117.3	600.2	1.24	4927.9	4930.3	2881.4	2.4051
6.5C	100.3%	117.5	602.68	1.29	4925.8	4929	2875.8	2.3991
6.5D	100.1%	117.81	600.95	1.23	4943.4	4945.7	2884.7	2.3985
6.5E	98.2%	117.45	599.71	1.3	4936.5	4938.7	2887.4	2.4065
65 Gyrations								
6.0A	91.3%	126	601.57	1.17	5033.6	5041.7	2912.4	2.364
6.0B	94.1%	122.24	601.08	1.22	5031.8	5039.4	2910	2.363
6.5A	96.3%	120.85	602.56	1.21	5044.9	5047.5	2938.8	2.3924
6.5B	97.9%	118.84	601.57	1.28	4975.8	4978.2	2903.8	2.3987
55 Gyrations								
6.5A	92.4%	125.12	601.82	1.26	5020.4	5029.6	2892.1	2.3487
6.5B	94.6%	122.24	601.82	1.25	5023.9	5028.6	2898.1	2.3581
7.0A	96.0%	122.29	603.55	1.23	5047.6	5050.7	2918.4	2.3672
7.0B	96.3%	120.3	602.68	1.29	5050.3	5054.2	2922.6	2.3693

Table 18. Rugby Project Specimen Samples

Specimen ID	Unc. Gmm	Height (mm)	V-Press	Angle	Dry (g)	SSD (g)	Wet(g)	Gmb
Project: NH-TRP-3-002(160)213: Devils Lake, Rugby								
75 Gyrations								
5.0A	93.0%	114.47	604.17	1.26	4676.9	4693.8	2707.9	2.355
5.0B	90.2%	118.28	604.17	1.21	4767	4792.3	2737.7	2.320
5.0C*	90.8%	117.5	604.67	1.3	4684.4	4719.3	2686.1	2.304
5.0D	91.1%	117.04	604.67	1.23	4715.6	4736.5	2704.5	2.321
5.5A	95.9%	115.19	602.44	1.25	4743	4752.4	2745.1	2.363
5.5B	94.7%	116.68	601.2	1.21	4755.5	4770.3	2745.1	2.348
5.5C*	92.9%	116.22	602.19	1.3	4721.9	4736.7	2715.6	2.336
5.5D	94.3%	114.42	604.92	1.25	4738.3	4744.5	2747.3	2.372
6.0A	93.4%	118.53	605.91	1.24	4728.6	4743.1	2696.3	2.310
6.0B	93.7%	118.22	603.92	1.2	4747.9	4761.4	2710	2.314
6.0C*	96.3%	114.47	601.69	1.31	4723.7	4729.9	2741	2.375
6.0D	95.1%	114.78	601.94	1.25	4737.1	4741.9	2746	2.373
6.5A	92.1%	119.67	603.92	1.31	4698.6	4704.6	2701.2	2.345
6.5B	93.0%	118.84	602.68	1.29	4716.6	4723.9	2700.7	2.331
6.5C*	95.2%	116.11	601.94	1.29	4728.5	4734.1	2721.6	2.350
6.5D	95.5%	115.03	602.91	1.24	4754.1	4757	2752.3	2.371
6.5E	55.3%	116.32	601.94	1.27	4831.1	4833.1	2803.5	2.380
6.5F	56.0%	114.83	600.95	1.25	4768.9	4770.9	2771.2	2.385
65 Gyrations								
6.0A	96.9%	119.15	602.44	1.2	4830.9	4843.8	2776.5	2.337
6.0B	97.7%	118.07	600.45	1.23	4851.2	4861.5	2804.7	2.359
6.5A	100.9%	115.91	600.95	1.26	4837.1	4841	2817.4	2.390
6.5B	98.7%	118.53	599.71	1.23	4911.1	4914.8	2846.7	2.375
55 Gyrations								
6.5A	98.9%	118.28	602.93	1.24	4832.6	4841.9	2781.1	2.345
6.5B	97.2%	120.33	601.69	1.21	4844.3	4857.4	2764.5	2.315
7.0A	99.6%	118.38	601.94	1.23	4858.4	4861.2	2796.6	2.353
7.0B	100.3%	117.56	601.69	1.43	4804	4809.6	2764.3	2.349

Table 19. Eddy Performance Tests

Performance Tests Samples									
Specimen ID	Gyrations	Unc. Gmm	Ht (mm)	V-Press	Angle	Dry (g)	SSD (g)	Wet(g)	Gmb
5.6A	35	91.4%	100	603.05	1.28	3981.2	4000	2268.2	2.299
5.6B	26	91.4%	99.9	602.81	1.34	3984	4004	2266.7	2.293
5.6C	25	91.4%	99.95	602.31	1.21	3982.4	4001.1	2258.8	2.286
6.1A	22	91.3%	99.85	603.3	1.38	3964.4	3980.4	2244.4	2.284
6.1B	27	91.3%	99.9	602.31	1.3	3984.2	4001.5	2262.9	2.292
6.1C	23	91.7%	99.9	600.83	1.35	3977.4	3992.9	2258.5	2.293
6.6A	25	92.0%	100	603.05	1.28	3973.9	3985.4	2251.7	2.292
6.6B	20	92.1%	99.9	602.81	1.37	3981.2	3992.6	2259.8	2.298
6.6C	17	92.0%	99.95	601.57	1.25	3978.2	3989.9	2249.1	2.285
5.2A	23	91.4%	99.85	600.83	1.25	3994.4	4024.6	2293.7	2.308
5.2B	23	91.3%	100	600.58	1.32	4001.7	4029.1	2300.8	2.315
5.2C	19	91.9%	100	601.57	1.29	3990.3	4014.7	2280.2	2.301
5.2D	21	91.3%	100	601.08	1.29	3997.9	4023.2	2287.9	2.304

Table 20. Rugby Performance Tests

Performance Tests Samples									
Specimen ID	Gyrations	Unc. Gmm	Ht (mm)	V-Press	Angle	Dry (g)	SSD (g)	Wet(g)	Gmb
5.9A	75	100.8%	116.47	603.79	1.26	4741.9	4753.2	2722.4	2.335
5.9B	75	98.9%	117.56	603.3	1.23	4713.1	4736.4	2686.6	2.299
5.9C	75	94.5%	118.84	607.25	1.21	4802.3	4813.3	2749.5	2.327
5.9D	75	94.1%	119.36	605.77	1.28	4821.3	4833.5	2756.5	2.321
5.9E	75	95.2%	119	605.28	1.24	4815.1	4826.3	2758.8	2.329
5.9F	75	95.6%	123.32	605.28	1.21	5064.3	5073.2	2926.1	2.359
5.9G	75	96.8%	112.25	604.78	1.25	4606.8	4612.6	2658	2.357
5.9H	17	76.8%	74.63	602.81	1.23	2798.9	2857.2	1572.1	2.178
5.9I	68	90.7%	74.94	602.56	1.31	2973.8	2990.8	1694.7	2.294
5.9J	53	91.1%	100	602.56	1.24	3963.6	3982.7	2252.2	2.290
5.9K	56	91.6%	100	603.3	1.24	3963.6	3983.1	2246	2.282
5.9L	40	91.7%	99.95	602.56	1.31	3967.2	3983.5	2248.5	2.287
6.1A	46	91.9%	99.95	603.79	1.24	3965.5	3982.4	2251.3	2.291
6.1B	83	91.7%	100	601.08	1.18	3971.5	3990.6	2256.4	2.290
6.1C	45	91.7%	100	603.34	1.27	3975	3990.1	2257.6	2.294
6.1D	38	91.7%	100	603.3	1.22	3954.3	3974.1	2241.1	2.282
6.6A	61	91.9%	100	603.55	1.22	3948.2	3961.3	2229.6	2.280
6.6B	44	92.0%	99.95	602.7	1.36	3936	3951.1	2212.8	2.264
6.6C	23	92.0%	99.95	601.37	1.25	3936.7	3948.2	2207.1	2.261
5.5A	24	91.3%	99.85	602.56	1.35	3993	4016.8	2281.4	2.301
5.5B	24	91.2%	99.95	602.31	1.32	3992.5	4017.7	2283.2	2.302
5.5C	23	91.3%	99.85	602.56	1.36	3991.3	4015.6	2283.1	2.304
5.5D	23	91.2%	100	601.33	1.32	3988.1	4015.5	2284.1	2.303