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LONG-LASTING OPEN GRADED FRICTION COURSE (OGFC) FOR SURFACING NEW MEXICO ROADS

Final Report

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SUMMARY PAGE

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14. Abstract

This study investigated the applicability of two performance grade (PG) binders: PG 64-28+ and PG 76-22+ in open graded friction course (OGFC) by comparing their performances to that of an existing PG 70-28+, which is the only binder currently being used in OGFC mixes in New Mexico. A goal of this study was to evaluate the effectiveness of New Mexico Department of Transportation (NMDOT)'s OGFC mix design, which is based on volumetric but not performance based. Considering the effects of aggregate gradation on OGFC performances, three different aggregate gradations (two 9.5 mm NMASs and one 12.5 mm NMAS) were employed in this study. A comprehensive laboratory experimental program was designed to investigate the drain-down, moisture damage, abrasion, aging, wet durability, and fracture performances of nine OGFC mixtures prepared with three binders and three aggregates. Based on the extensive analysis of the all the laboratory test results, it was found that PG 76-22+ OGFCs exhibits better performances than PG 70-28+ OGFCs. Therefore, this study recommends using PG 76-22+ binder in OGFC as an alternative to current PG 70-28+ binder for ensuring better performance in high traffic volume roads and in the southern (hot) regions of New Mexico. In addition, a binder grade zone map was developed in this study to help the pavement engineers to select the suitable binder for OGFC for different parts of the state. As per the evaluation of the NMDOT's current volumetric mix design method of OGFC, which is based on mainly tensile strength ratio (TSR) test, this study has involved several laboratory performance tests. Cantabro abrasion (CA) and semi-circular bending (SCB) tests were employed to characterize abrasion, aging, and fracture performances of an OGFC mixture. A Micro-Deval testing was used for evaluating the durability performance of an OGFC in wet condition. Based on these performance test results, it was observed that all three OGFCs meet the volumetric and TSR requirements, but PG 64-28+ OGFC performed poorly. It is suggested that the NMDOT's exiting method of OGFC design needs to include several performance tests to ensure adequate durability performance of an OGFC.

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PREFACE

The research reported herein is about finding suitable performance grade (PG) binders for open graded friction course (OGFC) mixes for New Mexico's traffic and weather conditions.

NOTICE

DISCLAIMER

ABSTRACT

This study investigated the applicability of two performance grade (PG) binders: PG 64-28+ and PG 76-22+ in open graded friction course (OGFC) by comparing their performances to that of an existing PG 70-28+, which is the only binder currently being used in OGFC mixes in New Mexico. A goal of this study was to evaluate the effectiveness of New Mexico Department of Transportation (NMDOT)'s OGFC mix design, which is based on volumetric but not performance based. Considering the effects of aggregate gradation on OGFC performances, three different aggregate gradations (two 9.5 mm NMASs and one 12.5 mm NMAS) were employed in this study. A comprehensive laboratory experimental program was designed to investigate the drain-down, moisture damage, abrasion, aging, wet durability, and fracture performances of nine OGFC mixtures prepared with three binders and three aggregates. Based on the extensive analysis of the all the laboratory test results, it was found that PG 76-22+ OGFCs exhibits better performances than PG 70-28+ OGFCs. Therefore, this study recommends using PG 76-22+ binder in OGFC as an alternative to current PG 70-28+ binder for ensuring better performance in high traffic volume roads and in the southern (hot) regions of New Mexico. In addition, a binder grade zone map was developed in this study to help the pavement engineers to select the suitable binder for OGFC for different parts of the state. As per the evaluation of the NMDOT's current volumetric mix design method of OGFC, which is based on mainly tensile strength ratio (TSR) test, this study has involved several laboratory performance tests. Cantabro abrasion (CA) and semi-circular bending (SCB) tests were employed to characterize abrasion, aging, and fracture performances of an OGFC mixture. A Micro-Deval testing was used for evaluating the durability performance of an OGFC in wet condition. Based on these performance test results, it was observed that all three OGFCs meet the volumetric and TSR requirements, but PG 64-28+ OGFC performed poorly. It is suggested that the NMDOT's exiting method of OGFC design needs to include several performance tests to ensure adequate durability performance of an OGFC.

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INTRODUCTION

SCOPE

The New Mexico Department of Transportation (NMDOT) uses open graded friction course (OGFC) material/layer for surfacing its high-volume traffic roads, however, it is found that the OGFC layers perform poorly in most of the cases. The OGFC is usually designed with an open graded aggregate to increase permeability, which allows faster removal of roadway surface water than traditional dense graded pavement and to enhance rough surface texture or skid resistance. Currently, the NMDOT uses only one performance grade binder: PG 70-28+ (or rubberized 70-28R+) all over the state irrespective of climatic conditions, whereas the PG binder selection should be location/climate specific. Since different PG binders are available in current days, it is required to evaluate the appropriateness of the available PG binders for the OGFC layers at different climatic regions. Furthermore, the NMDOT uses the traditional volumetric method for designing the OGFC layers without considering their performance. However, it is known that the volumetric mix design method performs well against rutting but poor against cracking. Therefore, there is a national trend where state DOTs are moving towards performance-based mix design, where an optimized mix is chosen based on both rutting and cracking. Studies have also showed that the OGFC layers are prone to have raveling distresses. More importantly, the OGFC is suspected to play significant roles in statewide top-down cracking problem in New Mexico. It is hypothesized that the high air content in OGFC leads to accelerated aging due to higher accessibility of air resulting in brittle OGFC that helps initiate cracking within thin OGFC layer and then easily propagate into the HMA surface below it. However, no attempt has been undertaken to prove this hypothesis. As such, the aforementioned concerns associated with the OGFC materials used in New Mexico were investigated in this study.

GOALS AND OBJECTIVES

The main goals and objectives of this research are to:

i. Examine whether NMDOT can remove the restriction on only one binder in OGFC.
ii. Determine if NMDOT can add few performance tests as per OGFC mix design.

REPORT ORGANIZATION

This report is subdivided into five chapters. Chapter 1 describes the scope and objectives of this research. Chapter 2 contains an in-depth literature review on the past research related to OGFC materials. It also covers the current practices of different state departments of transportation (DOTs) on OGFC materials. The rheological properties and performances of three different PG binders are presented in Chapter 3. Laboratory performances of OGFC mixes are summarized in Chapter 4. Finally, the observation and key findings are presented in Chapter 5.

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LITERATURE REVIEW

INTRODUCTION

A comprehensive literature review has been carried out to document the current state-of-the-practice regarding the use of open graded friction course (OGFC) materials by different state DOTs. Based on this review, this section summarizes the specifications and guidelines for OGFC mix design followed by other states. The finding of past researchers on OGFC materials are also discussed.

DEFINITION OF OGFC

The OGFC is a thin permeable asphalt concrete (AC) layer placed on a dense graded AC pavement. The OGFC is an open graded asphalt mixture that contains a high percentage of air voids than the traditional dense graded mix (*1*). Typically, the air void of a dense graded mix is restricted between 5% to 7%, whereas the recommended air void of an OGFC material lies between 15% to 22%. Figure 1 presents a schematic of an OGFC on top of a dense graded layer. The high air void is required to promote better permeability and friction characteristics. Furthermore, primarily coarse size aggregates with very little fines are used to prepare OGFC material to ensure a higher content of connected air voids (*2*). The OGFC is alternatively named, such as porous graded asphalt (PGA), porous friction course (PFC), and porous asphalt (PA) in other countries. It is usually paved as the final riding surface on roadways because of the safety and environmental benefits associated with it.

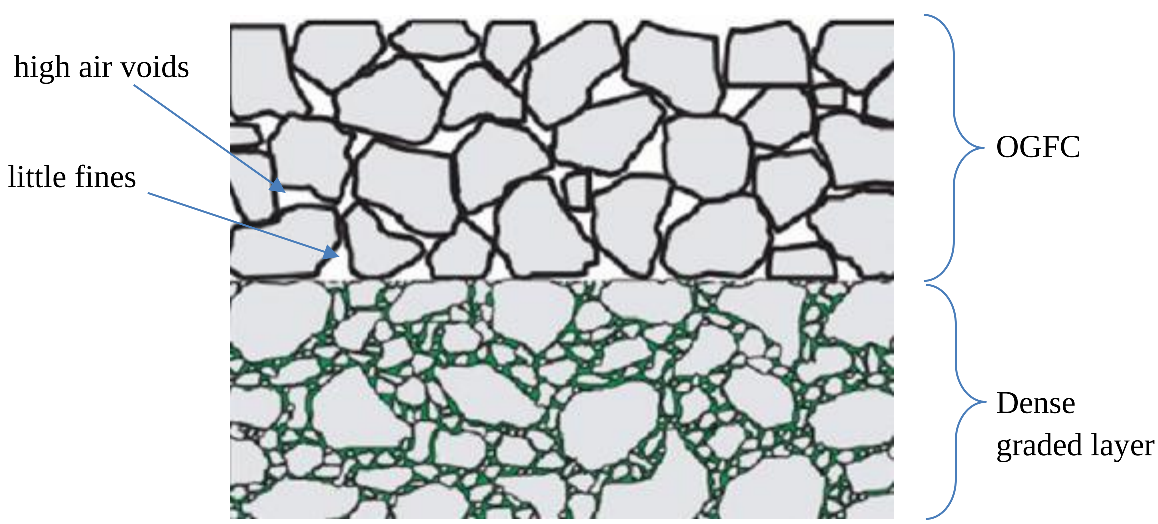

FIGURE 1 A schematic of OGFC on top of a dense graded layer

OVERVIEW OF OGFC USE IN THE UNITED STATES (US)

In the United States (US), OGFC was first introduced in the 1930s. Field trials of placing precoated aggregate chip seal coats using a paving machine in Oregon led to the invention of OGFC (*3*). In the early 1970s, Colorado provided one of the earliest documentations on the OGFC mix design procedure. It involved compacting test mixes in the lab and sawing the samples open to determine the amount of asphalt drain-down in the mixture. The goal was to seal the bottom third of the mixture to prevent water from penetrating the base layers. Another objective was to create an asphalt-rich layer that would adhere to the lower layers. In the mid-1970s, a mix design procedure was developed for OGFC based on the Hveem procedure to assess asphalt binder content and a drain-down test to determine construction-mixing temperature (*4*). The Federal Highway Administration (FHWA) developed a complete guideline for the OGFC mixture design procedure based on the Marshall mix design method (*5*). Later, National Center for Asphalt Technology (NCAT) released a new mix design method for the new generation OGFC in 2000 (*6*). Unlike the previous two methods, this mix design procedure utilizes the Superpave gyratory compactor to compact the specimen. This method improves the OGFC mix design procedure by including a) an improved selection measure for coarser aggregate gradation, b) the possibility of using polymer modified asphalt binders and fibers in the mixture, c) the use of air voids to determine optimum asphalt binder content, and d) the use of a mixture durability test.

OGFC gained popularity in the 1970s in response to the Federal Highway Administration (FHWA)'s program to increase frictional resistance on pavements (*7*). The wet weather benefits and enhanced pavement frictional performance of OGFC led to the rapid growth of their use across the country. As a result of this expansion, a number of failures began to occur because of the durability issues of OGFC. The durability problems of OGFC are raveling, drain-down segregation, and top-down cracking (*8-10*). The main disadvantage of these distresses is that once they begin, they progress rapidly. As a result, the OGFC has a shorter expected service life than the traditional dense mixes. For example, the expected service life of OGFC ranges between 7 to 10 years versus about 12 to 15 years for a dense graded mix (*11*). Besides the poor durability performance, maintenance of OGFC surfaces is also very difficult and costly (especially in cold regions). A variety of construction issues arose because of the binder drain-down problem. Therefore, the popularity of OGFC diminished over time because of its poor durability performance. Many states that had discontinued the use of OGFC on their pavements. However, increased usage of polymer modified binders in OGFC combinations, improved design methods, and improved construction control procedures (QC/QA) have resulted in a rapid resurrection of OGFC mixtures in warm climate states in the 2000s.

A number of reports are found that contain the national surveys on the use of OGFC mixtures by individual state highway agencies. The National Cooperative Highway Research Program (NCHRP) synthesis 180 (*12*) summarized the first significant national survey found in the literature. This report includes the results of a survey conducted in 1988. The survey received responses from 47 states, one territory, and three Canadian provinces. According to the report, 27 states were still using the OGFC mixture, while 21 states had stopped using it. The primary reasons given by the states for limited or discontinued use of OGFC mixtures are a) early failures, stripping of the underlying courses, b) construction difficulty, c) friction reduction, and d) loss of internal drainage characteristics with time. According to the report, the OGFC mixtures cost approximately 21% more than the dense graded mixtures. It also stated that, in general, state agencies do not consider the OGFC layer as a pavement structural layer. According to the report, one of the most severe causes of premature OGFC failures is the oxidation of the asphalt binder film.

The Transportation Research Board (TRB) published a report in December 1998 based on a national survey on the use of OGFC mixtures (*2*). This survey was conducted at a time when some states were transitioning from the original OGFC mixtures to the new generation mixtures. The survey received responses from 42 states, as shown in Figure 2. It reported that 19 states were using some type of OGFC mixture, four had never used the OGFC mixture, 19 had stopped using the mixture, and eight did not respond to the survey. According to the results of this survey, only Illinois and Wyoming were using OGFC in the cold state study area for this project. Table 1 lists the specific problems related to OGFC reported by the respective DOTs.

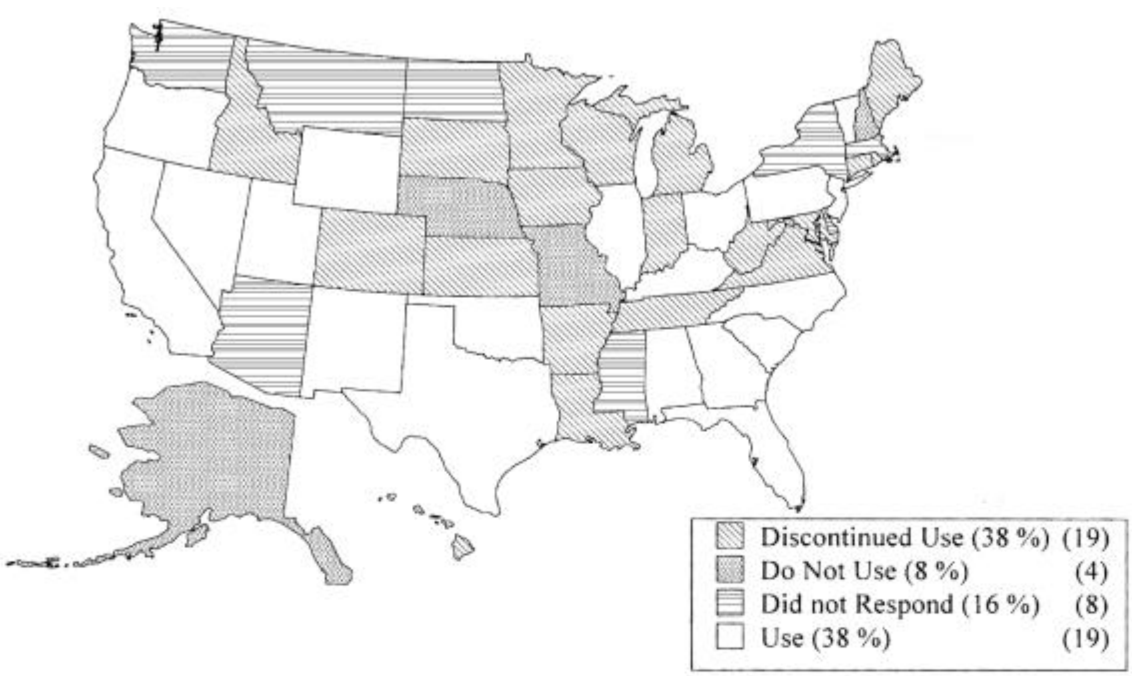

FIGURE 2 Use of OGFC by US states according to a survey by Kandhal and Mallick (*2*)

NCAT conducted another recent survey in 2015 (*7*). According to this survey report, only half of 41 responding agencies (40 states and Puerto Rico) use OGFC materials in their pavements. Figure 3 shows the use status of OGFC by different states according to their survey. The survey results revealed that agencies that do not use OGFC believed that their OGFC mix designs are inadequate to maintain the expected service life. The primary distresses that cause premature failures are raveling and top-down cracking.

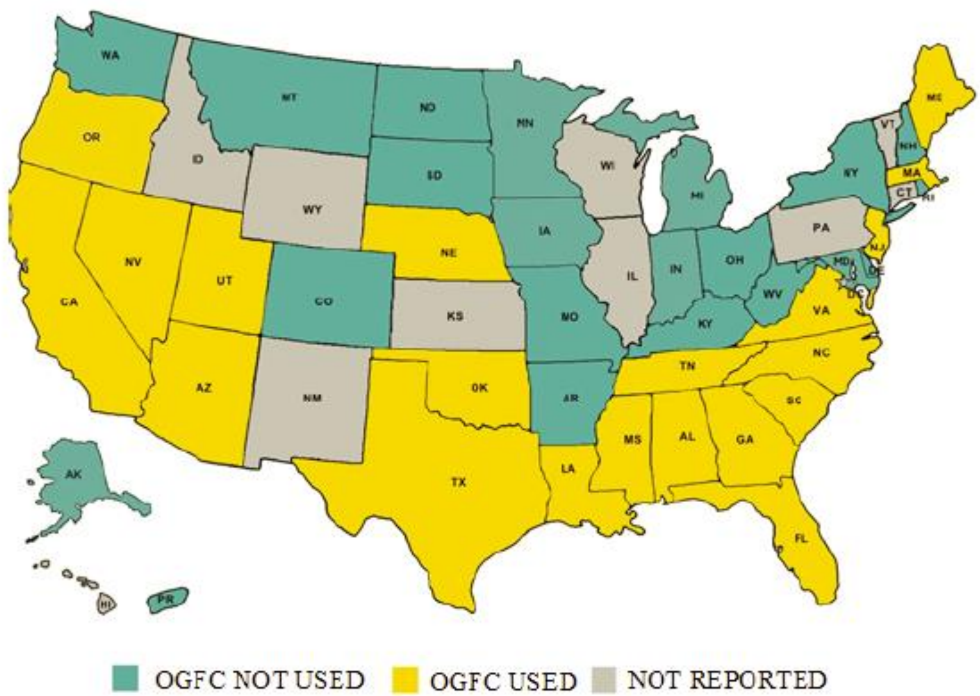

FIGURE 3 Use of OGFC by US states according to survey by NCAT 2015 (*7*)

TABLE 1 Problems related to OGFC reported by different states (*2*)

State code	Problems
IA	Removal of ice was very difficult.
MD	Raveling in OGFC.
ME	Removal of ice was very difficult.
MN	Deicing sand clogged voids; stripping of OGFC.
RI	Durability problem; widespread debonding; OGFC scraped by snowplows.
VA	Stripping in underlying layers; needed heavy fog coat after several years to prevent raveling.
AK	Filling up of voids, leading to moisture retention; prolonged freezing, and snow and ice removal problems.
LA	Extensive raveling.
TN	Stripping in underlying layers; aggregate loss in OGFC by raveling; snow and ice removal problem due to refreezing of melted snow and ice.
CO	Moisture damage to underlying layers.
ID	Sanding caused the filling up of voids.
KS	During winter snow and ice storm, voids became filled with water and froze; developed icy surface; took a substantially higher amount of salt to melt ice.
SD	Sand and salt plugged up the voids.
HI	Raveling because of absorptive aggregate.

BENEFITS OF OGFC

A survey was conducted by Huber et al. (*23*) to review the benefits of using OGFC on top of a dense graded layer. Responses were collected from all 50 state highway agencies in the United States and ten Canadian provinces. According to the respondents, increased frictional resistance is the most important benefit of utilizing OGFC, followed by driver visibility, pavement marking visibility, and noise reduction. The benefits of OGFC can be categorized into three groups: safety, economy, and environment.

Safety benefits

The use of OGFC can help to improve road safety, particularly in wet weather. The main advantage of OGFC is its excellent wet frictional resistance. Though the difference in skid resistance between OGFC and dense graded surfaces is not noticeable at a lower speed, it becomes significant when the speed is high. Many past studies reported that using OGFC improves frictional resistance and reduces accidents in wet situations (*13*).

The traditional dense graded layer is impervious and has poor drainage characteristics. As a result, rainwater can accumulate on the dense graded surface. This water accumulation diminishes tire-to-pavement contact, which can result in hydroplaning. This circumstance is particularly dangerous since it results in a lack of braking and steering control. These situations can be avoided by placing the OGFC layer on top of the dense graded layer that drains water from the surface (*14*).

Water spray refers to the tiny water particles ejected from rolling wheels of passing traffic on wet pavement surfaces. The coarser water particles formed when vehicles pass over ponds in poorly drained locations are known as water splashes. Both water spray and splash can reduce the driver's vision because the droplets formed by splash and spray have a higher density and larger size than fog droplets. These effects can also be minimized by using OGFC as it can drain water out from the surface quicker than the dense graded layer (*15*). Figure 4 depicts the differences in driving conditions between a road with and without OGFC.

a) Without OGFC b) With OGFC

FIGURE 4 Comparison of diving conditions on a pavement with and without OGFC (*7*)

The reduction of glare, especially at night, is another advantage of OGFC. OGFC can diffuse reflections, and this feature improves the visibility of road markers during both day- and night-time driving (*15*). In wet condition, the improvement is more evident because less water at the surface is associated with less light reflection (*16*).

Economic benefits

The use of OGFC improves the smoothness of pavement and increases diver's comfort. Past research shows that the usage of OGFCs on major roads can cut fuel consumption by about 2%. When the porous mixtures are compared to rougher mixtures, higher savings are also reported. Another cost-effective feature of OGFC is that it minimizes tire wear. This claim is based on a reduction in tire stresses caused by the better macro-texture of this type of mixture (*16*, *17*).

Environmental benefits

The ability of OGFC to minimize or manage highway noise levels is one of their most essential characteristics. In Europe, this has been a major motivator for its use on the pavement. The void structure of OGFC dissipates noise created by air pumping within tire threads. The sound tends to be absorbed rather than reflected by the spaces (*18*). When OGFC is compared to the regular dense graded layer, noise levels are projected to be reduced by 3 to 6 decibels. This is confirmed by numerous past studies, including data from numerous European countries and Canada and research findings in California (*19*). Compared to other pavement surfaces, OGFC was found to have the lowest noise level (*20*). Figure 5 compares the noise levels among different types of pavement surfaces. When a station wagon with radial cap tires was driven at 105 km/h, the sound levels were measured 15 m (50 ft) from the roadway (65 mph) (*2*). Higher driver comfort levels can be achieved using OGFC since noise reduction is perceived not only outside the vehicle but also inside.

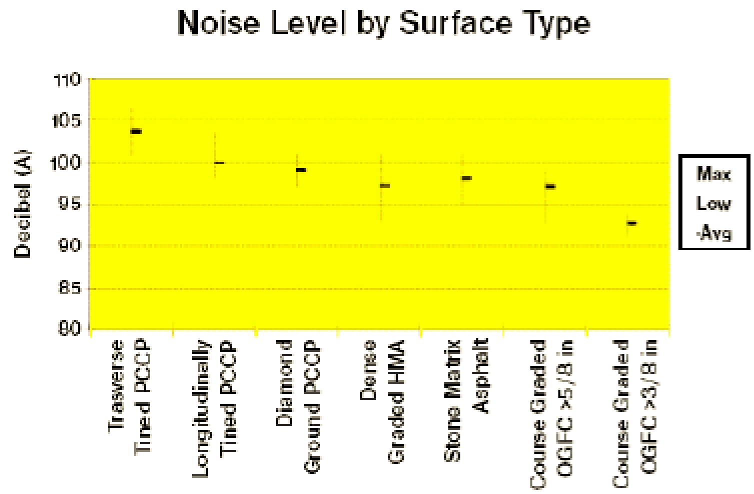

FIGURE 5 Comparison of noise levels among different pavement surfaces (*21*)

OGFC is also proven to minimize the amounts of numerous contaminants in stormwater runoff in recent research. Total suspended particles, total metals, and chemical oxygen demand are claimed to be lower in an OGFC runoff than in a traditional dense graded layer runoff (*22*). The inter-connected void structure of OGFC can act as a stormwater filter.

DRAWBACKS OF OGFC

Although these safety and environmental benefits are appealing, the use of OGFCs is also associated with a number of drawbacks. Poor durability, reduced permeability and noise reduction capacity, high construction cost, unfavorable winter maintenance, and minimal structural contribution are the main drawbacks related to the use of OGFC.

Poor durability

Durability is the main problem for which many state DOTs discontinued using OGFC in their pavement. It is reported that OGFC has a shorter service life than the traditional dense graded material. The average service life of an OGFC is around 7 to 10 years, whereas the average service life of a traditional dense graded layer is around 12 to 15 years (*11*).

Raveling is the most common cause of OGFC's durability issues, shown in Figure 6. Raveling is the most common cause of OGFC's durability issues. Raveling is defined as the loss of aggregates at the surface of the pavement caused by repeated abrasion by traffic and often aggravated by the presence of moisture. It quickly worsens the pavement condition once it starts. Different state DOTs have reported a variety of experiences. Mix design (particularly material selection) and construction issues are to blame for many of the OGFC's raveling problems. Binder drain-down is another reason for raveling issue. The asphalt binder has a tendency to leach from the aggregate skeleton due to the open graded structure of the mix, and this situation is known as the binder drain-down problem (*23*). As the binder drains from the aggregates, the aggregates near the surface of the layer become progressively less and less coated. It becomes prone to raveling under combined actions of traffic, oxidation, and embrittlement. To avoid drain-down issues and binder component degradation, some states limit the mixing temperature.

FIGURE 6 Raveling problem associated with OGFC (*7*)

Since the OGFC has higher binder content, it is prone to have drain-down segregation (*24*), and the problem becomes severe when the used binder has a lower viscosity. The top-down cracking occurs mainly due to the traffic and weather action. It is known that the top surface of the pavement becomes brittle due to the oxidation effect. The truck tires cause a high surface horizontal tensile stress that can initiate cracking on the top surface. These cracks later propagate downward; therefore, they are known as top-down cracking. The high air content in OGFC leads to accelerated aging due to higher accessibility of air resulting in brittle OGFC that helps initiate cracking within the thin OGFC layer and then easily propagate into the surface below it.

Reduced permeability and noise reduction capacity

If OGFC material is not durable, its functionality also diminishes. When the OGFC material breaks, the broken particles clog the pores. The clogged pores drastically reduce the permeability and noise reduction capacity of pavement. In Denmark, frequently cleaning and construction in the two-layer system are used to improve the functional characteristics of the mixture over a long period of time (*25*).

Higher construction cost

The construction cost of OGFC is relatively higher than the traditional dense graded material (*23*). The reason for this higher cost is the use of the highest quality materials (e.g., aggregate, binder, and fiber). The cost per ton of OGFC in the United States is between 10 and 80 percent higher than the cost of traditional dense graded material. However, the service life is between 50 and 100 percent of the traditional dense graded material.

Unfavorable winter maintenance

Winter maintenance of OGFC is a big challenge. Because OGFCs have a poorer thermal conductivity than dense graded material, they freeze quicker and for longer periods than standard pavement surfaces (*7*). Snow plowing is not recommended for OGFC because the blades of the snowplow can damage the surface. Spreading sand and small aggregates to increase frictional qualities under freezing conditions might be difficult since these materials fill the voids, reducing drainage and noise reduction capabilities (*15*). Therefore, deicing agents are used for winter treatments of OGFC pavements. The two most prevalent ice control chemicals for OGFCs are calcium chloride and sodium chloride (*8*). However, frequent application of larger amounts of deicer agents and higher care are required to make the OGFC surface functional during the winter. These requirements increase the maintenance costs for OGFC.

Minimal structural contribution

OGFC is typically considered to have no or minimal contribution to the pavement's structural system (*2*). However, it adds to the expense of a paving system.

OVERVIEW OF OGFC MIX DESIGN PROCEDURES

In this section, different mix design procedures for OGFC materials currently used in the US are discussed. It should be noted that there is not a unified method adopted by all state DOTs for designing OGFC. Different DOTs use different criteria for defining the optimum binder content of OGFC.

Federal Highway Administration (FHWA) method

The FHWA issued Technical Advisory T5040.31 in December 1990, which included complete guidelines for the OGFC mixture design procedure (*5*). The methodology is based on the Marshall mix design. It has four design steps: estimation of surface capacity and optimum binder content, selection of aggregate gradation, determination of mixing temperature, and evaluation of moisture damage susceptibility.

Estimation of surface capacity and optimum binder content

In this step, the surface capacity of the predominant aggregate fraction is determined by immersing and draining the material in S.A.E. No. 10 lubricating oil, in accordance with AASHTO T270. The material that passes through a 19.0 mm sieve and is retained on a no. 4 sieve is considered the predominant aggregate fraction. After that, the binder content is calculated using an empirical formula that takes into account the surface constant value and apparent specific gravity of the predominant aggregate. Table 2 lists the granular aggregate selection criteria.

TABLE 2 FHWA specifications for granular aggregates

Parameter	Specified Value
Los Angeles (LA) abrasion	< 40%
Fractured faces	> 75% for particles with two faces, > 90% for particles with one face
Mineral filler's specifications	AASHTO M 17

Selection of aggregate gradation

In this step, the percentage of fine aggregate by weight of total aggregate is calculated, taking into account the volume of asphalt and the design air voids content (suggested at 15%). If the size of the coarse aggregate voids is insufficient to contain the asphalt and air voids, the coarse aggregate gradation needs to be changed. Table 3 shows the FHWA recommended aggregate gradation.

TABLE 3 FHWA recommended aggregate gradation for OGFC

Sieve opening (mm)	Percent passing
12.5	100
9.5	95-100
4.75	30-50
2.36	5-15
0.075	2-5

Determination of mixing temperature

In this step, the optimum mixing temperature is determined by conducting a Pyrex glass plate test and keeping in view the drain-down potential. This test can be carried out by spreading approximately 1000 grams of coated aggregate on a Pyrex glass plate under specified conditions. After 60 minutes, the bottom of the glass plate is examined. A slight puddle of asphalt at the points of contact between the aggregate and the glass plate is preferable, and the test is repeated at a higher or lower temperature.

Evaluation of moisture damage susceptibility

In this step, the resistance to water needs to be evaluated using an immersion-compression test according to AASHTO T 165 and AASHTO T 167.

National Center for Asphalt Technology (NCAT) method

The NCAT released a mixture design method for the new generation OGFC in 2000 (*6*). This method has been improved over time from subsequent research. The NCAT design procedure has four major steps: materials selection, selection of aggregate gradation, determination of optimum binder content, and evaluation of moisture susceptibility.

Materials selection

In this step, both granular aggregate and binder need to be selected such that they meet their respective criteria. Table 4 lists the required specifications for selecting granular materials.

TABLE 4 Specifications for granular aggregates

Parameter	Specified Value
Los Angeles (LA) abrasion	< 30%
Fractured faces	> 90% for particles with two faces, 100% for particles with one face
Flat and elongated particles	< 5 and 20% (ratios of 5:1 and 3:1, respectively)
Fine aggregate angularity (FAA)	> 45

Table 5 summarizes the criteria for binder selection based on expected traffic volume. It is possible to use cellulose fiber or mineral fiber in proportions of 0.3 and 0.4% by weight of the total mixture, respectively. The typical fiber content ranges from 0.2 to 0.5% and is determined by the results of the mixture drain-down test.

TABLE 5 Specifications for asphalt binder

Recommended Type of Binder	Volume Traffic
High stiffness binders made with polymers, fiber addition is desirable	Medium to high
Polymer modified binders or fiber addition	Low to medium

Aggregate gradation selection

In this step, a gradation is selected such that it can ensure the high voids content in the total mixture and the existence of stone-on-stone contact in the coarse aggregate skeleton. The coarse aggregate is defined as the fraction of aggregate that is larger than no. 4 sieve. It is hypothesized that the mixture will have stone-on-stone contact when the volume of voids in the coarse aggregate of the compacted mixture (VCA_{mix}) (that will be filled with air, the effective asphalt content, and fine aggregate) is smaller than the volume of voids in the coarse aggregate calculated from the dry rodded unit weight (VCA_{DRC}) using only the coarse aggregate, this is $VCA_{mix} < VCA_{DRC}$, the mixture will have stone-on-stone contact. The NCAT recommended aggregate gradation is shown in Table 6.

TABLE 6 NCAT recommended aggregate gradation for OGFC

Sieve opening (mm)	Percent passing
19.0	100
12.5	80-100
9.5	35-60
4.75	10-25
2.36	5-10
0.075	2-4

Determination of optimum binder content

In this test, the optimum asphalt content is selected based on the application of different laboratory tests using compacted and uncompacted samples with different binder contents. Table 7 summarizes these tests and specification limits. All compacted specimens need to be fabricated using 50 gyrations in the Superpave gyratory compactor. In addition, a permeability value greater than 100 m/day (328.08 feet/day) is desired; however, the determination of permeability in compacted specimens is optional.

TABLE 7 Tests to determine optimum binder content

Test	Specification
Air void	≥ 18%
Drain-down loss	≤ 0.30%
Cantabro abrasion loss (unaged condition)	≤ 20%
Cantabro abrasion loss (aged condition)	≤ 30%

Evaluation of moisture damage susceptibility

In this step, the moisture susceptibility of the design mixture is evaluated through the tensile strength ratio (TSR) test (AASHTO T 283). The specified TSR value is a minimum of 80%.

OGFC mix design practice of state DOTs

As mentioned before, there is not a unified method adopted by all state DOTs for designing OGFC. Many states have developed their own specifications using a combination of FHWA, NCAT, AASHTO, and/or ASTM guidelines.

Aggregate gradations

Table 7 shows the aggregate gradations specified by different state DOTs (*8*). It can be seen that all the states primarily use either 9.5 mm or 12.5 mm nominal maximum aggregate size (NMAS) aggregates, and the majority of them are gapped between the 9.5 mm and the 4.75 mm sieves.

Only Oregon uses a gradation with a 19.0 mm NMAS aggregate for its OGFC materials. All listed gradations listed in Table 8 do not have fines lower than 2.36 mm sieve except no. 200. The NCAT recommended gradation consists of 12.5 mm NMAS aggregate, whereas the FHWA recommended consists of 9.5 mm NMAS aggregate.

TABLE 8 Aggregate gradations recommended by different states (*2*, *7*, *26*)

State	25.0 mm	19.0 mm	12.5 mm	9.5 mm	4.75 mm	2.36 mm	0.075 mm
AL		100	85-100	55-65	10-25	5-10	2-4
AZ			100		30-45	4-8	0-2
CA			100	78-89	28-37	7-18	
DE			100	88-98	25-42	5-15	2-5
FL			100	85-100	10-40	4-12	2-5
GA		100	85-100	55-75	15-25	5-10	1-4
IL		100	90-100	30-50	10-18		2-5
LA		100	85-100	55-75	15-25	5-10	1-4
MS			100	80-100	15-30	10-20	2-5
NE		100	95-100	40-56	20-30	5-12	0-3
NC		100	85-100	55-75	15-25	5-10	2-4
NV			100	90-100	35-55	5-18	0-3
NM			100	90-100	25-55	0-12	0-4
OK		100	85-100	35-65	10-25	5-10	2-4
OR		99-100	90-98	25-40		2-12	1-5
OR2	99-100	85-96	55-71	15-30		5-15	1-6
SC		100	85-100	55-75	15-25	2-20	0-2
TN		100	85-100	55-75	10-25	5-10	2-4
TX		100	80-100	35-60	1-20	1-10	1-4
UT			100	92-100	36-44	14-20	2-4
VT			100	95-100	30-50	5-15	2-5
WY			100	97-100	25-45	10-25	2-7
NCAT		100	80-100	35-60	10-25	5-10	2-4
FHWA			100	95-100	30-50	5-15	2-5

Asphalt binders

Both neat and polymer modified binders have been used with OGFC mixes. However, the use of modified binders is becoming more common because they effectively increase the service life of OGFC pavements and prevent drain-down loss. The styrene-butadiene-styrene (SBS) polymer and rubber (e.g., crumb rubber modifier and ground tire rubber) are the most commonly used polymer modifiers in the US. These modifiers can make an asphalt binder stiffer for OGFC mixtures, increasing cohesion in the aggregate stone skeleton. As a result, OGFC mixes containing modified asphalt binders typically have higher resistances to rutting, cracking, and raveling damage, resulting in more extended durability in the field. Table 9 shows the different binder grades (or types) specified by different state DOTs (*7*, *8*, *26*). The most commonly used binder grade is PG 76-22.

Stabilizing agents

Stabilizing additives are often used to extend the life of OGFC mixtures by preventing drain-down and increasing tensile strength. Cellulose and mineral fiber are the most commonly used stabilizers. They are typically added to the mixture at a rate of 0.2-0.5 percent by total weight (*8*).

Fillers or anti-stripping agents

Different fillers or antistripping agents are recommended for OGFC to improve the bond between aggregates and asphalt binders and to prevent moisture damage to the mixtures. Hydrated lime, limestone dust, and liquid anti-stripping agent are common fillers or anti-stripping materials used in OGFC mixtures.

TABLE 9 Asphalt binder grade recommended by different states (*7, 8, 26*)

State	Binder grade
AL	PG 76-22
FL	PG 76-22M, PG 76-22HP (HP: high polymer)
NB	PG 58-28+ rubber
NV	PG 76-28M (M: modified)
AZ	PG 64-22+ rubber, PG 58-22+ rubber
MS	PG 76-22
VA	PG 76-22
GA	PG 76-22
LA	PG 76-22M
TX	PG 76-XXM
SC	PG 64-22, PG 76-22
OK	PG 76-22HP
NM	PG 70-28+, PG 70-28R+
NC	PG 64-22, PG 76-22M
ME	PG 76-28M
WS	PG 70-28M, PG 76-28M

OVERVIEW OF OGFC CONSTRUCTION PRACTICES

Although the production of OGFC uses the same equipment and processes as dense graded material, the production of OGFC layers demands some specific considerations.

Production

The production of OGFC, like that of dense graded materials, needs specific attention to aggregate moisture control. Some states demand the use of aggregate in a dry surface condition for OGFC production because it allows for better temperature control and a more homogenous mixture when the aggregate has low variability and low moisture content (*23*). OGFC sometimes requires the integration of fibers and modified binders, which can be accommodated with conventional asphalt plants. The use of any of these items, however, requires the installation of a fiber feed mechanism (*13*). The fibers are released and mixed with the aggregate in the pugmill of a batch plant or the drum of a drum plant when the asphalt binder is melted. Dry, loose fibers are often added with specific equipment fluff the material to a predetermined density and then blow a precise amount into the mixing plant. Fiber can be introduced into a drum plant by blowing fiber into it continuously (within 1 foot of the asphalt binder line). Bags of fiber can be dropped directly into the pugmill in batch plants. The fiber is spread throughout the liquid as the bags melt (*13*).

To improve the fiber distribution, the mixing times should be increased when production is done in a batch plant. Drying time should also be increased, as lower temperatures are stipulated (in comparison to other combinations' production temperatures), resulting in longer aggregate drying times and lower plant production rates. Because OGFC is characterized by drain-down susceptibility, which can be worsened by excessive temperature during production, temperature control is especially important. Some states limit the mixing temperature to avoid drain-down issues and binder component degradation. To avoid drain-down difficulties, the FHWA recommends keeping the binder viscosity in the range of 700 to 900 centistokes while determining the mixing temperature (*5*).

Storage and transportation

It is also required to keep mixture storage and transportation times as minimum as possible because of OGFC's drain-down problem. Some state DOTs have set maximum storage times between one and twelve hours (*23*). FHWA suggested that the combined handling and hauling of the OGFC mixture should be limited to 40 miles or 1 hour. Tarps are necessary to avoid the crusting of OGFC mixtures during transportation. Insulated truck beds for OGFC transportation are required by some state DOTs.

Surface treatment

OGFC should not be used to repair profile distresses or any other type of structural distress. The pavement surface should be fixed prior to OGFC construction to avoid zones that allow water to accumulate (e.g., zones with permanent deformation) because this negatively affects not only the OGFC layer but also the underlying pavement layers. To ensure appropriate water discharge from the OGFC, lateral and longitudinal drainage of the underlying layer must be supplied. OGFC should be placed over an impermeable layer to prevent vertical water seepage problems in underlying layers.

Placement

To provide a smooth surface, the paver should constantly move with few stops, and a remixing material transfer mechanism should be considered (*13*). Because surface depressions are more difficult to rectify with OGFC than with dense graded layer, it is vital to minimize mixed delivery with cold lumps and avoid jarring the paver when using direct delivery from the truck to the paver. In addition, auger extensions are recommended when using asphalt pavers with extendible screeds to avoid uneven mixture distribution between the paver's center and edge (*23*). The use of a hot screed in an asphalt paver is recommended to avoid over-pulling the material and to reduce the need for raking, which can result in areas with fewer voids or, more likely, uneven void distribution across the pavement. Raking can also result in an ugly surface texture and poor aesthetics, which are difficult to roll out with compaction.

Compaction

Because OGFC is normally manufactured with modified binders and is frequently laid at lower thicknesses than a dense graded layer, special attention to placement and compaction temperatures is required. Thin layers cool faster and require less compaction time. OGFC is frequently created in thin layers of 20 to 25 mm (0.75 to 1 inch) thickness in the United States, whereas PA is typically made with a layer thickness of 40 to 50 mm (1.57 to 1.97 inch) in Europe (*23*). In the United States, however, the standard layer thickness for the latest generation OGFC is 32 mm (1.25 inches) (*7*).

The most frequent method of compacting OGFC mixes is with static steel-wheel rollers (*13*, *23*). On thin layers (20 mm), two to four passes with an 8- to 9-ton tandem roller (within the required temperature range) are usually sufficient to complete the compaction operation. The FHWA advises compacting OGFC with one or two passes of an 8- to 10-ton static steel-wheel roller (20). Heavy rollers (over 10 tons) should be avoided for OGFC because they can cause excessive aggregate breakage, and pneumatic-tired rollers should be avoided because their kneading action lowers mixture drainage capacity by blocking surface pores (*23*).

OVERVIEW OF OGFC CONSTRUCTION PRACTICES

Maintenance is a fundamental aspect to consider in any project involving OGFC since these activities cannot be performed in the same way as for conventional dense graded material.

Winter maintenance

Specific winter maintenance techniques are required by OGFC. Winter maintenance of an OGFC requires more salt (or deicing agents) and more frequent applications than a dense graded surface (*23*). Deicing products, followed by liquid deicer agents, and sand, are now the most effective winter treatments. To increase traction, the FHWA recommends developing snow and ice control utilizing chemical deicers and plowing rather than employing abrasive materials (20). Spreading sand to increase friction and speed deicing contributes to void clogging, reducing drainage and noise reduction capabilities, two of the most critical OGFC benefits (5). Because deicers can flow into an OGFC rather than remaining on the surface, the Oregon Department of Transportation has proposed research on organic deicers with higher viscosity and electrostatic charge technology (similar to that used in emulsified asphalt) to improve deicer bonding on the surface (*24*).

Surface maintenance

Fog seals are used in only a few states to do preventive maintenance. Although there is no quantitative data on the impact of these treatments, fog seals are believed to extend the life of porous mixtures because they offer a thin layer of unaged asphalt at the surface (*5*). Fog seal should be applied in two passes (at a rate of 0.05 gal/yd^2) using a 50 percent dilution of asphalt emulsion without any rejuvenating chemicals, according to the FHWA. Research in Oregon regarding permeability reduction and changes in pavement friction on certain OGFC pavements generated by fog seals concluded that the mixtures still retain porosity and keep the rough texture related to their capability to reduce the potential for hydroplaning.

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EVALUATION OF BINDERS FOR OGFC

INTRODUCTION

This chapter documents the work done in this study to evaluate different PG binders for OGFC material. The tested binders are PG 64-28+, PG 70-28+, and PG 76-22+, respectively. The binder grades are selected based on weather conditions in New Mexico. All three binders were collected from Holly Frontier Inc. The rheological properties were determined using a dynamic shear rheometer (DSR) at the UNM laboratory. In addition, the permanent damage and fatigue performances of these binders were determined. Because polymer modification condition affects the overall performance of a binder, the presence of polymer in the tested binders were also verified to be consistent with the binder type.

SELECTION OF ASPHALT BINDER GRADES

The primary objective of this research is to evaluate the possibility of selecting binder grades for OGFC based on temperature data. Therefore, this study investigated the temperature profile of New Mexico, and hence it divided the state into different temperature zones in the most suitable groups. Finally, the binder grades were selected for each temperature zone.

Location and climatic condition of New Mexico

New Mexico is the fifth-largest state in the United States, covering 121,412 square miles and lying primarily between 32° and 37° north latitude and 103° and 109° west longitude (*27*). The topography of the state is primarily made up of high plateaus or mesas, with several mountain ranges, canyons, valleys, and typically arid arroyos. About 4,700 feet above sea level is the average elevation. The state map is shown in Figure 7. The yearly average temperature varies from 64 degrees Fahrenheit in the extreme southeast to 40 °F or lower in the high mountains and valleys of the north. However, elevation is a more important element in influencing a location's temperature than latitude. During the warmest months (for example, July), average monthly maximum temperatures range from slightly above 90°F at lower elevations to slightly above 70 °F at higher elevations. During the summer, though, individual afternoon temperatures at elevations below 5,000 feet frequently approach 100 °F. Minimum temperatures below freezing are prevalent in all parts of the state during the coldest months (e.g., January). During the winter, typical daytime temperatures range from above 50 °F in the southern and central lowlands to 30 °F in the northern higher altitudes. In the southern valleys, the freeze-free season lasts more than 200 days, whereas in the northern highlands, it lasts less than 80 days. The average annual precipitation throughout much of the southern desert is less than 10 inches, but more than 20 inches at higher elevations in the state. New Mexico has a lot of sunshine (75 to 80 percent). Because of the lower mountain temperature, average relative humidity is lower in the lowlands but greater in the highlands. The relative humidity level varies from near 65 percent at daybreak to approximately 30 percent in the middle of the afternoon. Over the state, wind speeds are normally modest, though occasionally severe winds accompany frontal activity in the late winter and early spring months.

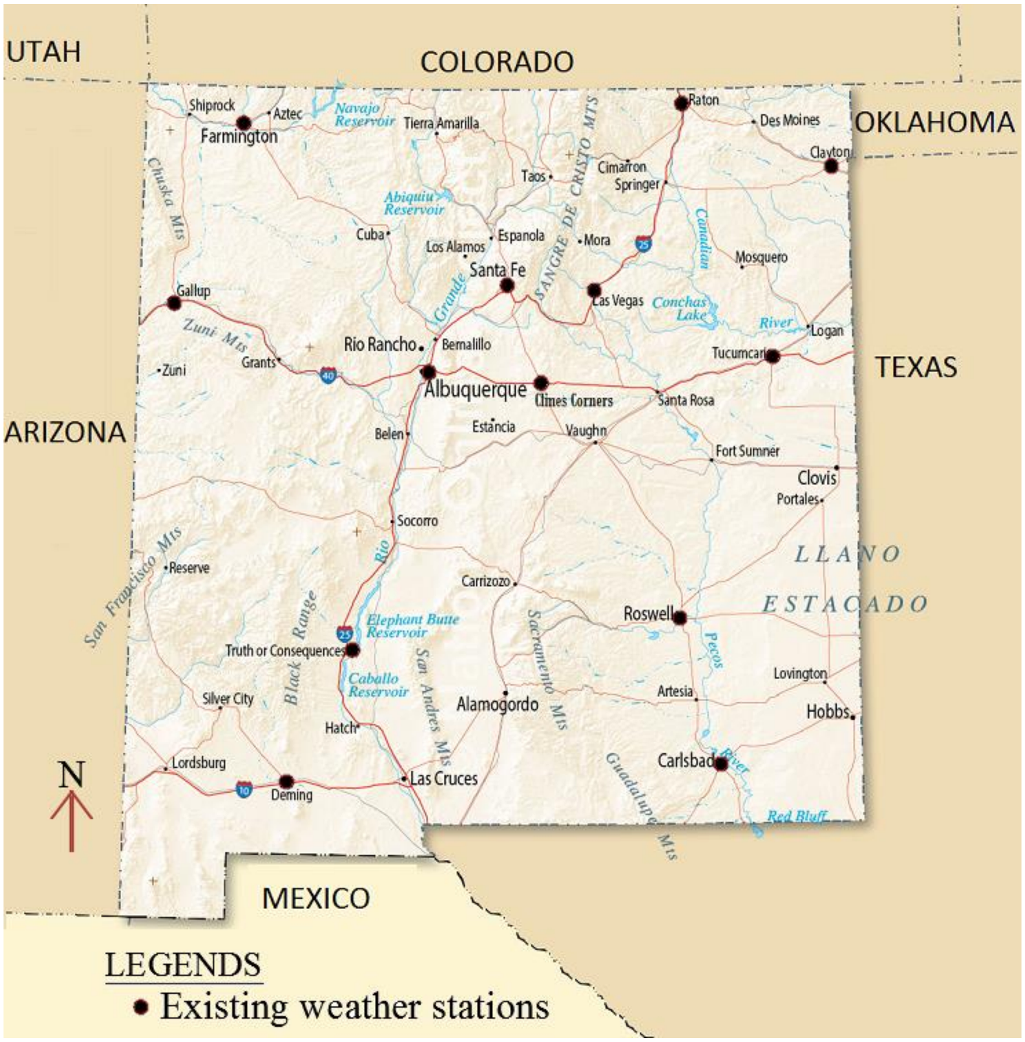


FIGURE 7 Map of New Mexico (*27*)

Long-term pavement performance (LTPP) data

To evaluate the pavement temperature variation within the state, this study used the long-term pavement performance (LTPP) data. The LTPP data was downloaded from the LTPP website. Figure 8 shows the locations of the data stations and their required high temperature grades at 98% reliability. It illustrates that the required high temperature grades of all stations can be divided into four different temperatures (52, 58, 64, and 70 °C). The coolest stations (52 °C) are found in northern mountain regions. The following temperature stations (58 °C) are located mostly in the state's northern and central-west parts. The warmer stations (64 °C) are distributed in the central, east, and northwest parts. The hottest stations (70 °C) are located in the southern parts of the state.

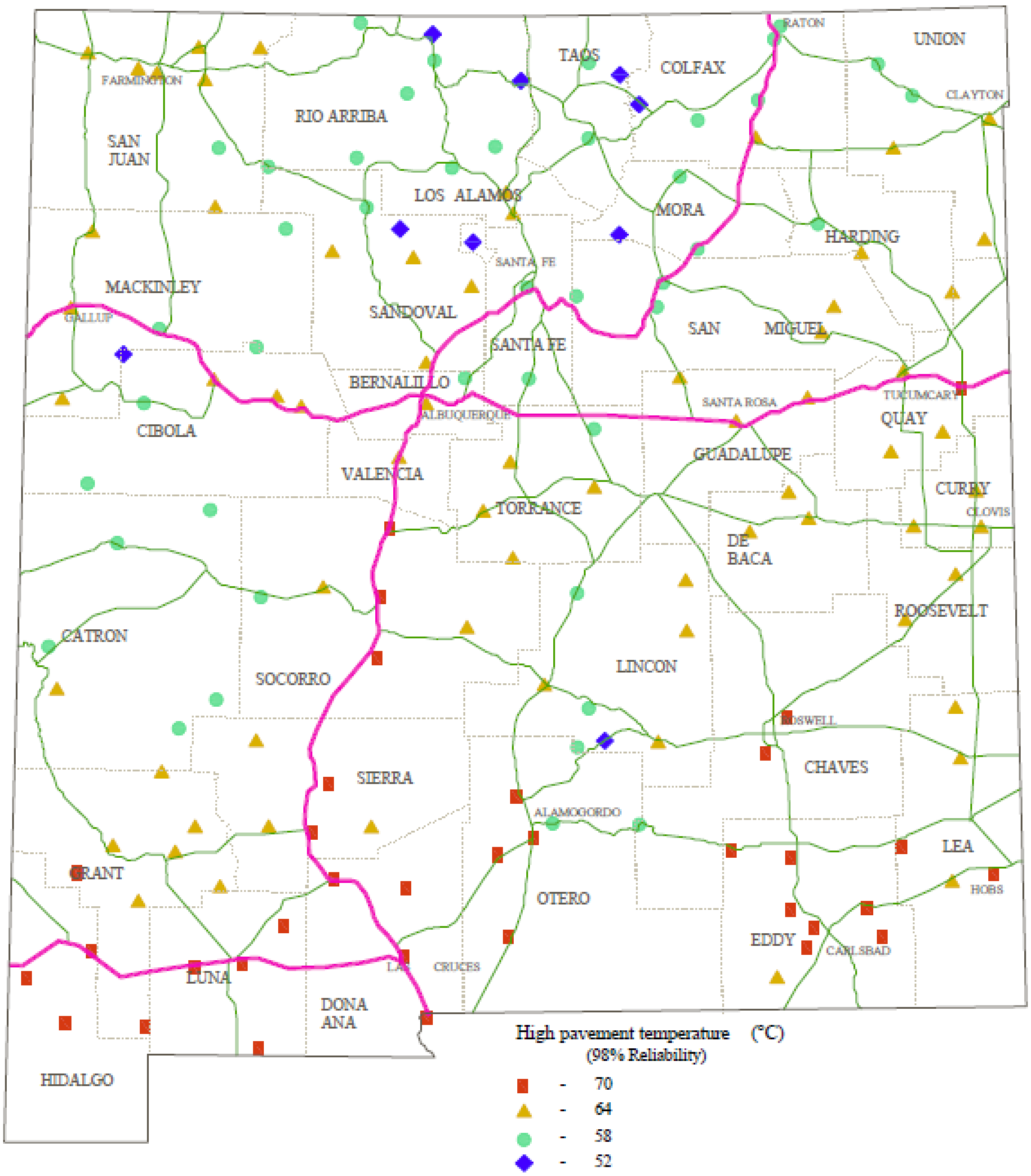


FIGURE 8 LTPP temperature stations for New Mexico

Proposed temperature zones and binder grades

Because few stations require a high temperature grade of 52 °C, this study divided the state into three temperature zones: 58, 64, and 70 °C. The temperature zones are shown in Figure 9. After that, binder grades were selected for each zone based on its high temperature requirement. However, instead of using exact required high temperature grades, this study proposed the next upper binder grades for each temperature zone to avoid drain-down. For example, if the required high temperature grade of a temperature zone is 58 °C based on LTPP data, the proposed binder grade for OGFC is 64 °C. As such, the proposed binder grades are PG 64-28+, PG 70-28+, and PG 76-22+ for colder, warmer, and hotter regions, respectively.

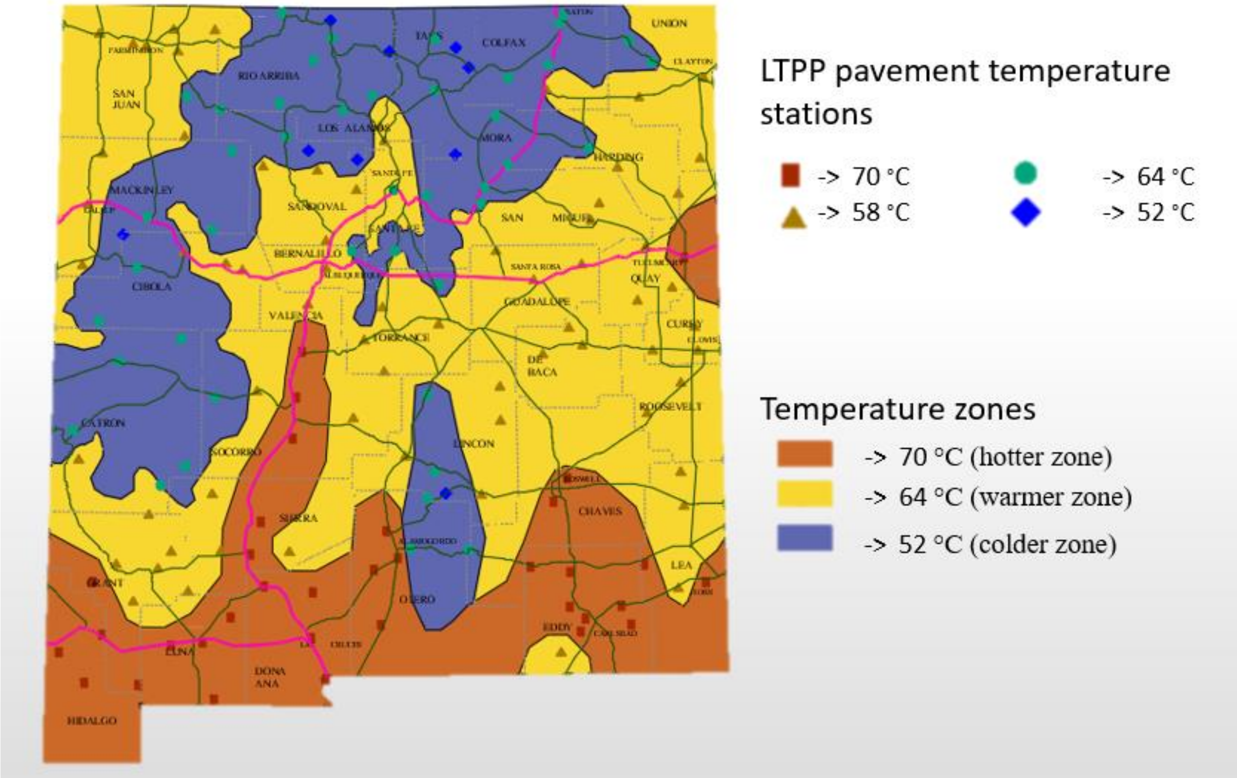


FIGURE 9 Temperature zones and proposed binder grades

VERIFICATION OF PRESENCE OF POLYMER

Three different binders of three proposed grades were collected for this study. Because polymer modified binders are effective in minimizing the drain-down loss, NMDOT specifies to use polymer modified binders in OGFC material. The presence of polymer in the collected binders was verified in this study. According to past literature, Fourier transform infrared (FTIR) spectroscopy is the most suitable way to detect the presence of polymer in a binder (*28*). This study used an attenuated total reflection (ATR) FTIR, as shown in Figure 10, to examine whether the binder is polymer modified or not.

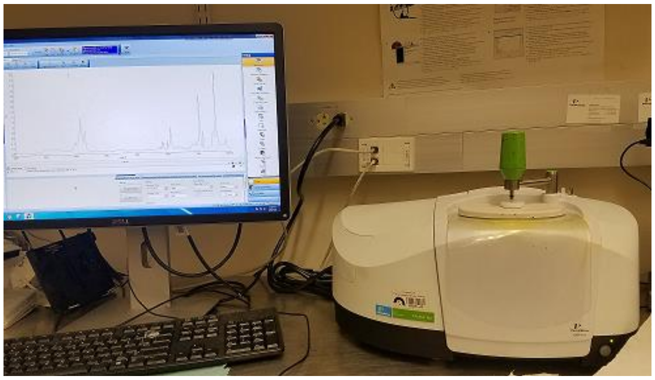


FIGURE 10 FTIR device used in this study (*28*)

Description of FTIR Spectroscopy

The Fourier transform infrared (FTIR) spectroscopy is a technique widely used in organic chemistry to identify the presence of different functional groups in a sample (solid, liquid, or gas) based on its energy absorption of infrared light. The working mechanism of an FTIR device is depicted in Figure 11. A beam splitter directs light from a source to the FTIR device. The light is split into two beams by the splitter. One beam is directed at a stationary mirror, while the other is directed at a moving mirror. The beams then return to the splitter to recombine. The moving length moves to produce constructive and destructive interferences (an interferogram). The recombined light then passes through an attenuated total reflection (ATR) prism and, as a result, the sample. The sample absorbs different wavelengths of light depending on the vibrations of the different functional groups that exist, and the remaining light is detected by a detector. The detector then reports light intensity versus time for all wavelengths at the same time. Finally, with the assistance of a computer, a mathematical function known as a Fourier transform is used to convert an intensity versus time spectrum into an intensity versus frequency spectrum. FTIR spectroscopy takes advantage of the fact that molecules absorb specific frequencies based on their chemical structure. These absorbed frequencies are known as resonant frequencies because they correspond to the transition energy of the vibrating bond or group. The bond energy for different functional groups varies depending on the shape of the molecular potential energy surfaces, the masses of the atoms, and the associated vibrionic coupling. As a result, each functional group has its own FTIR spectrum.

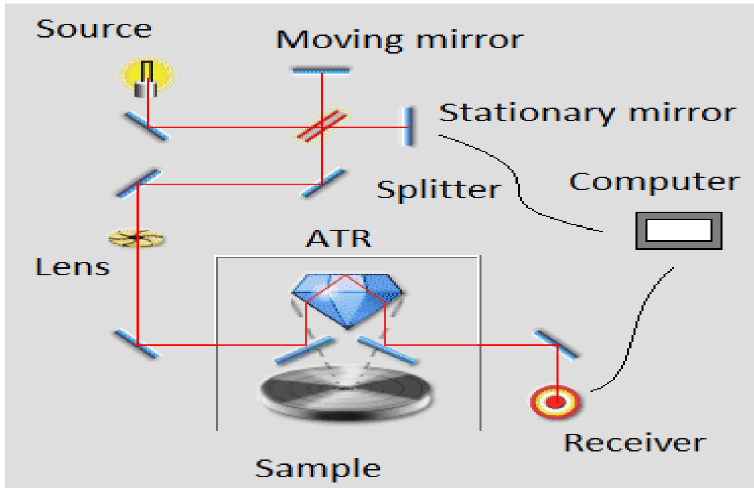
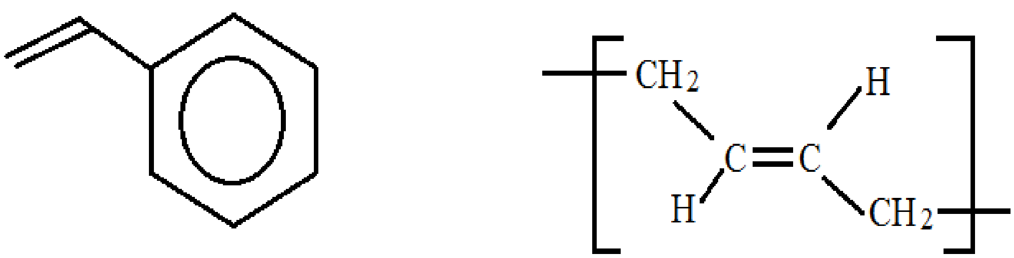


FIGURE 11 FTIR analysis mechanisms (*28*)

Chemical structures of different functional groups and FTIR wavenumbers

To utilize the FTIR analysis, it is essential to know the chemical structure of the polymer. The most common polymers used for modifications are styrene-butadiene (SB), styrene-butadiene-styrene (SBS), or styrene-butadiene-rubber (SBR). Thus, the presence of polymer can be easily detectable if there are FTIR peaks (or spikes) for polystyrene and polybutadiene copolymers. The chemical compositions or structures for polystyrene and polybutadiene are shown in Figure 12.



a) Styrene b) Butadiene

FIGURE 12 Chemical structure of SBS polymer (*29*)

The aromatic compound polystyrene (also known as ethenylbenzene) has the chemical formula $C_6H_5CH{=}CH_2$. As shown in Figure 12(a), it is a mono-substitute benzene with one hydrogen atom replaced by an ethenyl functional group. As a mono-substitute benzene, polybutadiene produces two distinct absorption peaks at wavenumbers 699 cm^{-1} and 757 cm^{-1}, respectively *(29*). Butadiene, on the other hand, has the chemical formula $H_2C{=}CH{-}CH{=}CH_2$. The polybutadiene molecule has a trans-alkene structure, as shown in Figure 12(b), and it produces a peak at wavenumber 966 cm^{-1}. However, it can be a terminal mono-substitute alkene and produces two peaks at 911 cm^{-1} and 993 cm^{-1}, respectively. A sample absorption spectrum of SBS polymer is depicted in Figure 13.

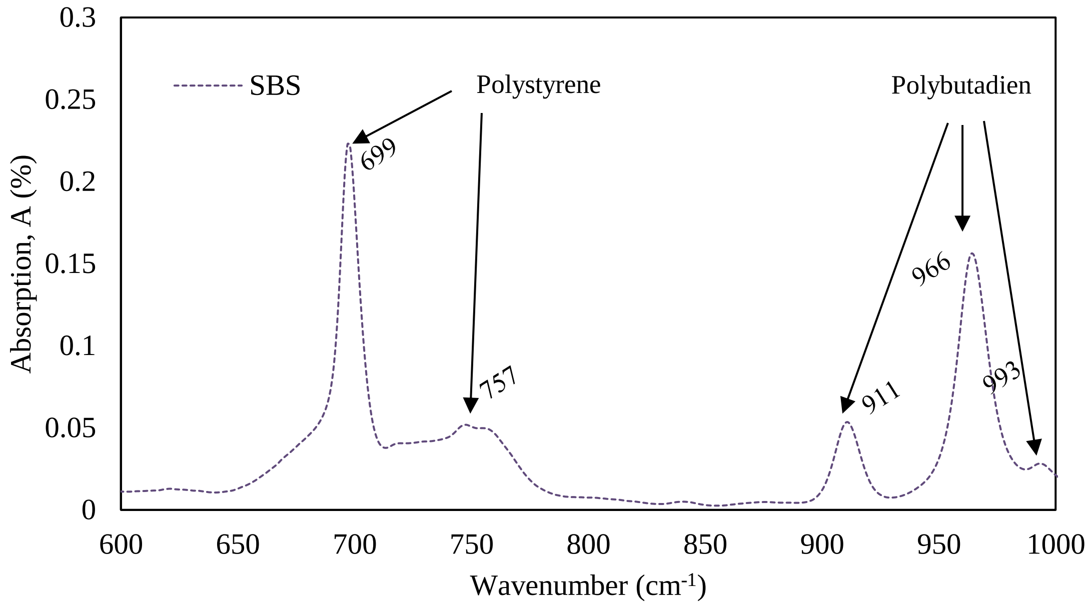
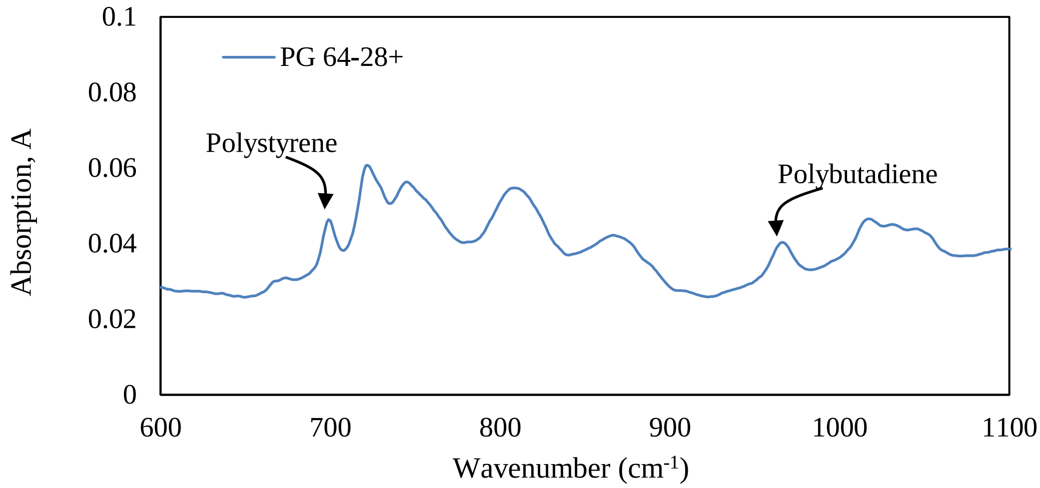


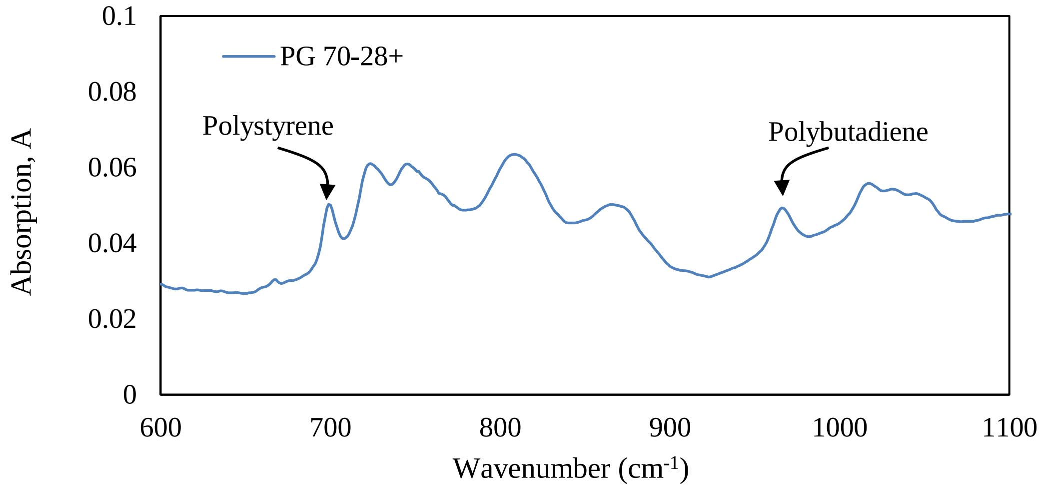
FIGURE 13 FTIR spectrum for SBS polymer (*30*)

FTIR Spectra for the tested binders

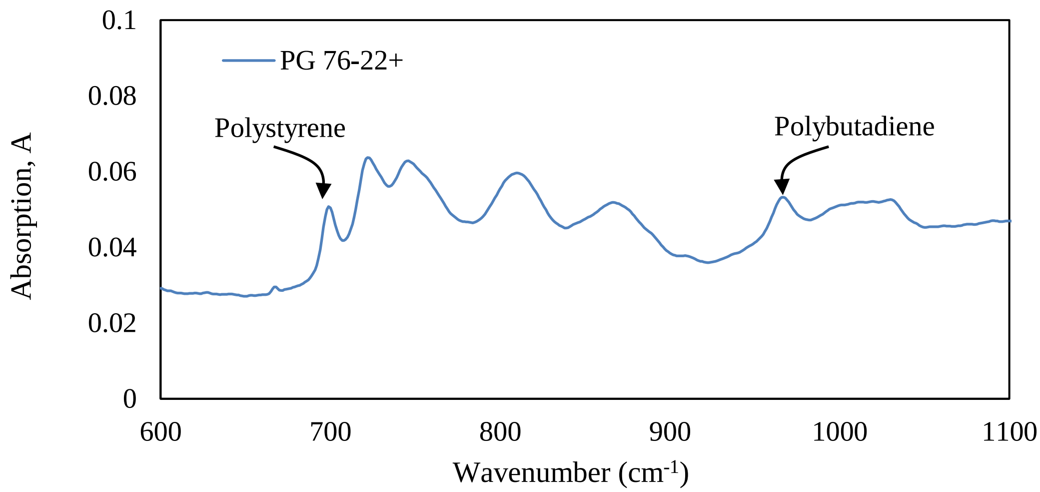
Figure 14 shows the generated FTIR spectra for all three tested binders. Figure 14(a) shows that PG 64-28+ binder has two small peaks at wavenumbers 699 cm^{-1} and 966 cm^{-1}, which indicates the presence of styrene and butadiene polymers in the binder. Thus, it can be said that PG 64-28+ binder is a polymer modified binder. Similarly, two peaks at wavenumbers 699 cm^{-1} and 966 cm^{-1} can also be observed for PG 70-28+ and PG 76-22+ binders, as shown in Figures 14(b) and 14(c). Thus, both PG 70-28+ and PG 76-22+ are also polymer modified binders.



a) PG 64-28+



b) PG 70-28+



c) PG 76-22+

FIGURE 14 FTIR spectra for tested binders

RHEOLOGICAL CHARACTERIZATION

The rheological property of an asphalt binder represents its viscous and elastic behaviors at different times and temperatures. This study evaluates the viscoelastic properties of tested binders using the DSR test.

Binder rheology background

Asphalt binder is a rheological viscoelastic material. The stiffness of an asphalt binder under dynamic loading is known as the complex modulus, *G**. It represents resistance to deformation when a binder is subjected to repeated shear loading. It is a time and temperature dependent property that has two parts: storage modulus, *G'* to represent the elastic part, and loss modulus, *G"* to describe the viscous part, as shown in Figure 15. At any given time (or frequency), *G** is termed as a combined form of G' and G'' as shown in Eq. (1) through Eq. (4). Phase angle. δ is a time dependency indicator to represent the time lag between applied stress and resultant strain. It can also be expressed as a combination of G' and G'' as shown in Eq. (5). The range of δ is between 0º to 90º, where 0º represents a pure elastic solid and 90º is a viscous liquid.

$$G^*(\omega) = G'(\omega) + iG''(\omega) \tag{1}$$

$$G'(\omega) = G^*(\omega) \cdot \cos\delta \tag{2}$$

$$G'(\omega) = G^*(\omega) \cdot \sin\delta \tag{3}$$

$$\left|G^*(\omega)\right| = \sqrt{(G'(\omega))^2 + (G''(\omega))^2} \tag{4}$$

$$\delta = \tan^{-1}(G''(\omega) / G'(\omega)) \tag{5}$$

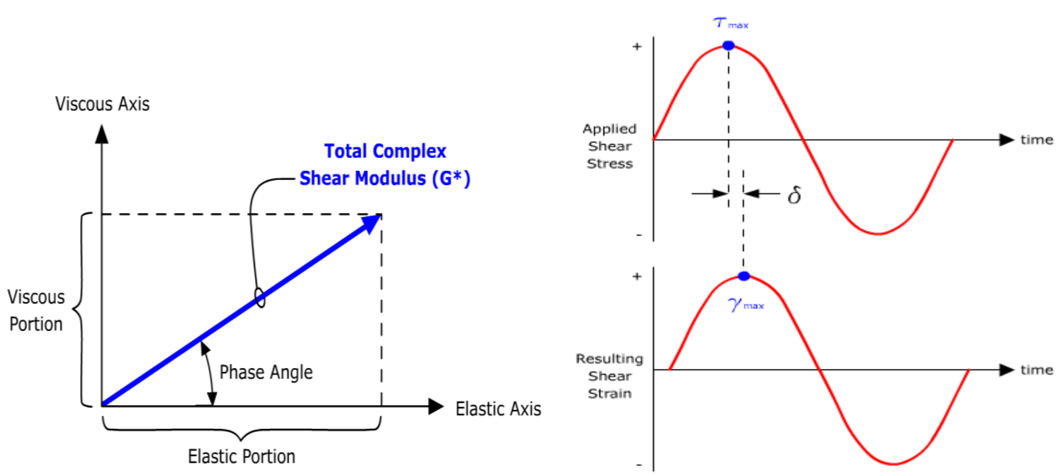


FIGURE 15 Viscoelastic behavior of asphalt binder

Aging simulation

In a practical scenario, the asphalt pavement (hence asphalt binder) is subjected to a different degree of aging due to oxidation and other weather effects. Thus, it is required to evaluate the rheological properties of an asphalt binder at different aging levels. Typically, a short-term aging occurs during mixing binder with aggregate at an elevated temperature at the mixing plant. In the laboratory environment, this type of aging is simulated through rolling thin film oven (RTFO) test. During the service life of a pavement, the paving materials get continuously pressurized through carrying of truck loading and oxidized through percolated oxygen. This long-term aging is simulated in the laboratory environment using a pressure aging vessel (PAV).

Rolling Thin Film Oven (RTFO) Aging

In RTFO aging, 35 gm of asphalt binder is poured into an RTFO bottle and rotated immediately to make a thin film of the asphalt binder. After that, the bottle is placed in the RTFO chamber, and the thin film of the asphalt binder is subjected to continuous heating (163^{o}C) and airflow (4000 ml/min) for 85 minutes. In this study, the short-term aging of all binders was done using an RTFO in the binder laboratory at UNM, as shown in Figure 16(a). The RTFO aging was conducted according to AASHTO T 240.

Pressure Aging Vessel (PAV) Aging

In PAV aging, 50 gm of short-term aged sample is poured into a PAV can. The binder sample is then subjected to heat at 100^{o}C temperature (simulates moderate climate) and air pressure of 2.10 MPa for 20 hours in a sealed PAV. The test temperature is varied in terms of the climatic region where asphalt is aged. The heating temperature is specified as 90^{o}C, 100^{o}C, and 110^{o}C for a cold, moderate, and hot climate, respectively. In this study, the long-term aging of all binders was simulated using a PAV in the binder laboratory at UNM, as shown in Figure 16(b). The PAV aging was conducted according to AASHTO R 28.

a) RTFO aging b) PAV aging

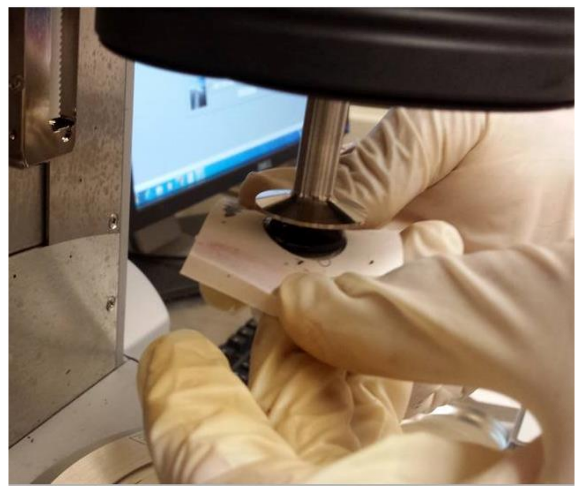
FIGURE 16 Binder aging procedures

Dynamic shear rheometer (DSR) test

Test descriptions

The dynamic shear rheometer (DSR) test, as shown in Figure 17, is used to characterize the viscous and elastic behavior of an asphalt binder at medium to high temperatures. From this test, G* and δ of a binder are determined at different temperatures. It is known that the rheological property is dependent on the applied loading rate; thus, a constant oscillating strain rate of 10 rad/s is specified by the AASHTO 315. Furthermore, AASHTO 315 also specifies the sample dimensions based on the aging level. For unaged and RTFO aged conditions, the diameter of the specimen is 25 mm, and the height is 1 mm, shown in Figure 17(b). For PAV aged condition, the diameter of the specimen is 8 mm, and the height is 2 mm, shown in Figure 17(c). In this study, the DSR test was performed on the tested binders at different temperatures and at different aging conditions.

a) DSR device



b) Sample in 25 mm parallel plates

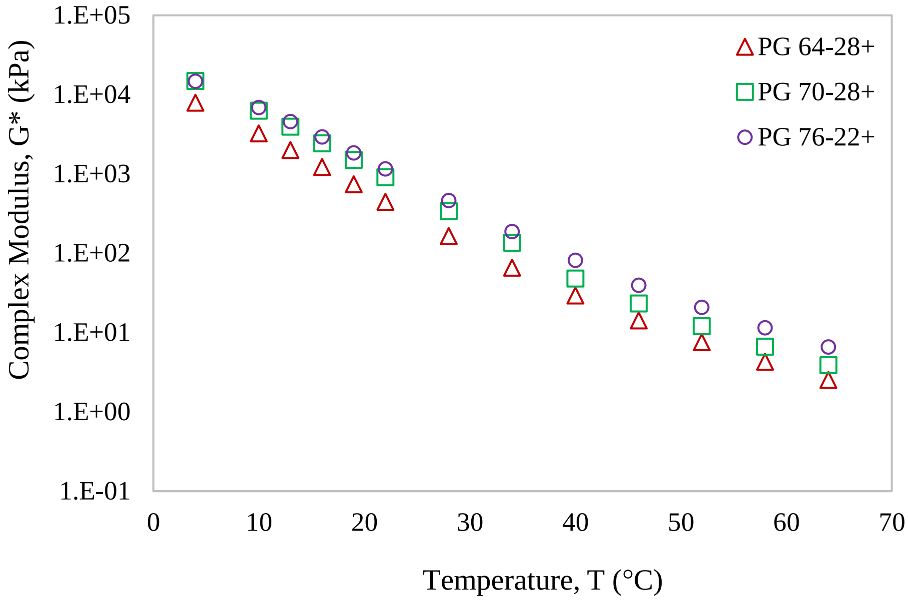
c) Sample in 8 mm parallel plates

FIGURE 17 Dynamic shear rheometer (DSR) test setup

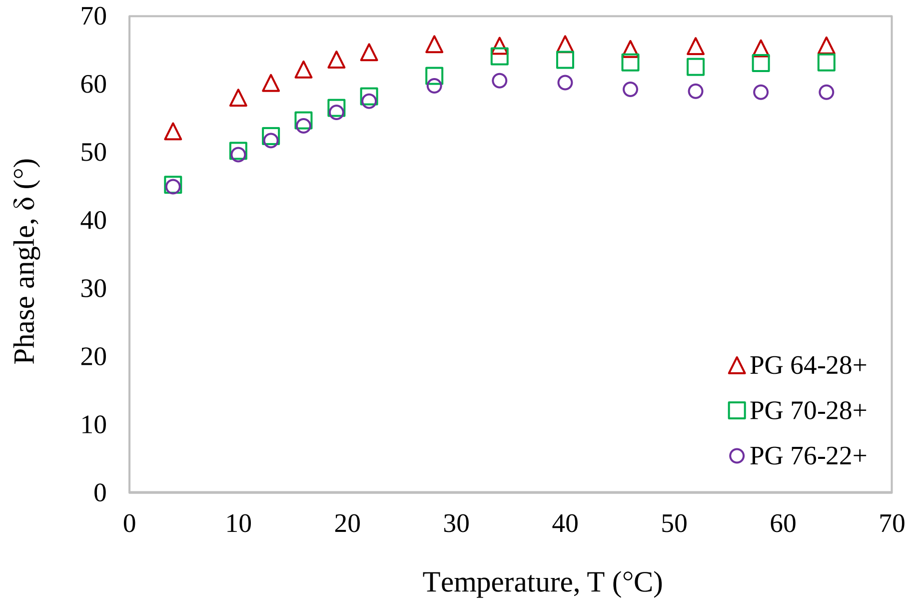
Test results

The DSR test results for the unaged binders are shown in Figure 18. Figure 18(a) compares the complex modulus values of the tested binders at different temperatures. It can be seen that the complex modulus is the highest for PG 76-22+ binder regardless of the temperature. On the other hand, PG 64-28+ binder has the lowest complex modulus among the tested binders. At lower temperatures, PG 70-28+ binder has complex modulus values comparable to PG 76-22+ binder. However, the complex modulus of PG 70-28+ binder becomes lower than PG 76-22+ binder at higher temperatures. It should be noted that the higher complex modulus indicates higher resistance to deformation. Thus, from these results, it can be said that PG 76-22+ binder is less susceptible to loading deformation than the other two binders. Based on the same logic, PG 64-28+ binder is the opposite properties. Figure 18(b) compares the phase angles of the tested binders at different temperatures. PG 76-22+ binder has the lowest phase angles, and PG 64-28+ binder has the highest phase angles regardless of the temperature. At lower temperatures, PG 70-28+ binder has phase angles comparable to PG 76-22+ binder. However, the phase angle of PG 70-28+ binder becomes higher than PG 76-22+ binder at higher temperatures. As a lower phase angle represents the elastic response of the binder, PG 76-22+ has better elastic properties than the other two binders, and PG 64-28+ binder has the lowest elastic property.

Figure 19 shows the DSR test results for the RTFO-aged. The complex modulus values of the tested binders at different temperatures are given in Figure 19(a). Like the unaged condition, the complex modulus in the RTFO-aged condition is the highest for PG 76-22+ binder regardless of the temperature. On the other hand, PG 64-28+ binder has the lowest complex modulus among the tested binders. PG 70-28+ binder has slightly lower complex modulus values than PG 76-22+ binder for the entire temperature range. Figure 19(b) compares the phase angles of the tested binders at different temperatures. PG 76-22+ binder has the lowest phase angles, and PG 64-28+ binder has the highest phase angles regardless of the temperature. PG 70-28+ binder has greater phase angles than PG 76-22+ binder. The DSR test results for the PAV-aged binders are shown in Figure 20. Like the unaged and RTFO-aged conditions, the PAV-aged binders have similar complex modulus and phase angle rankings.

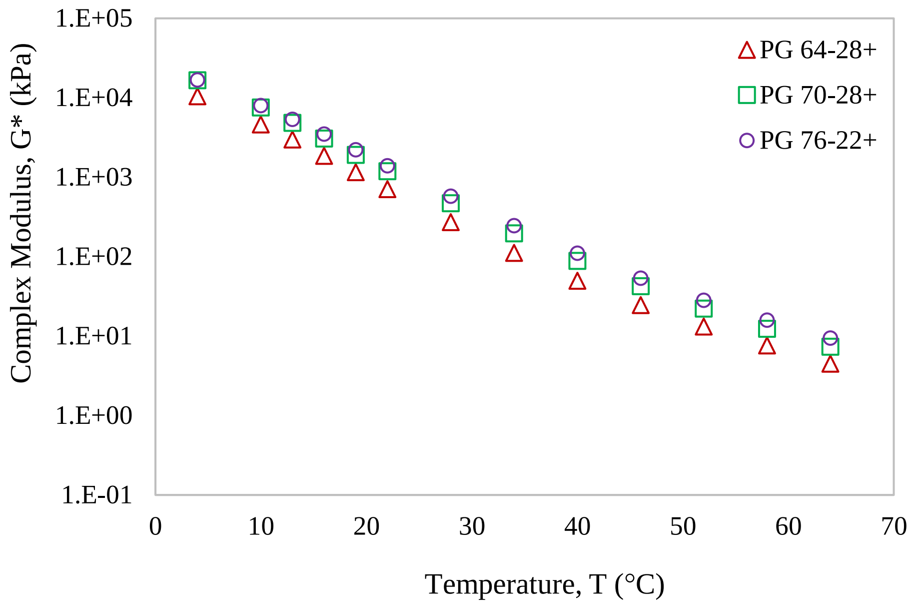


(a) Complex modulus

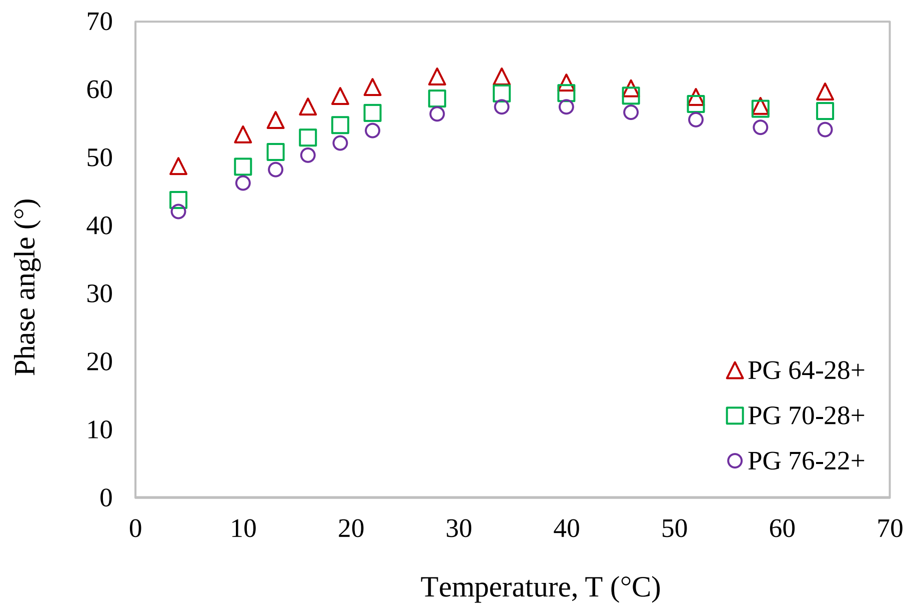


(b) Phase angle

FIGURE 18 DSR test results for unaged binders

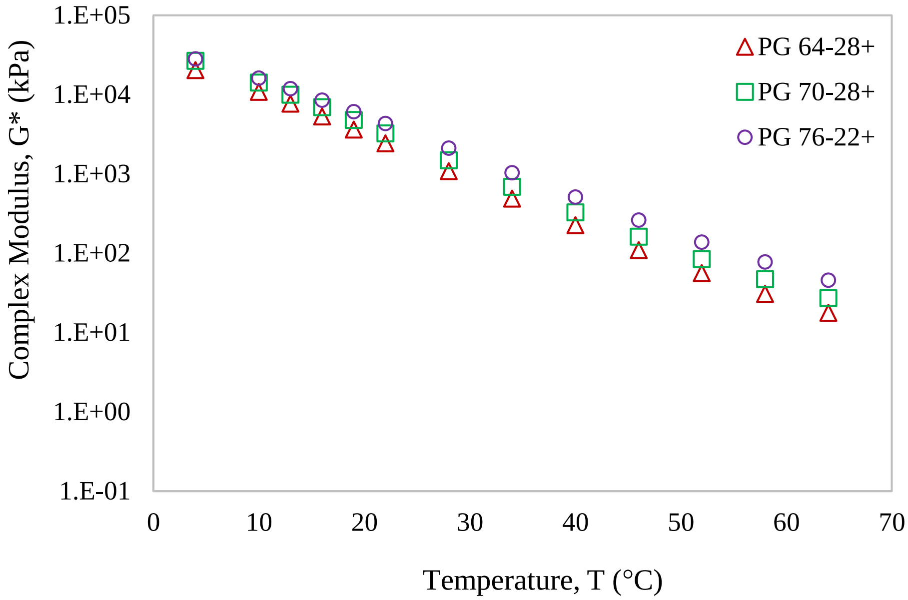


(a) Complex modulus

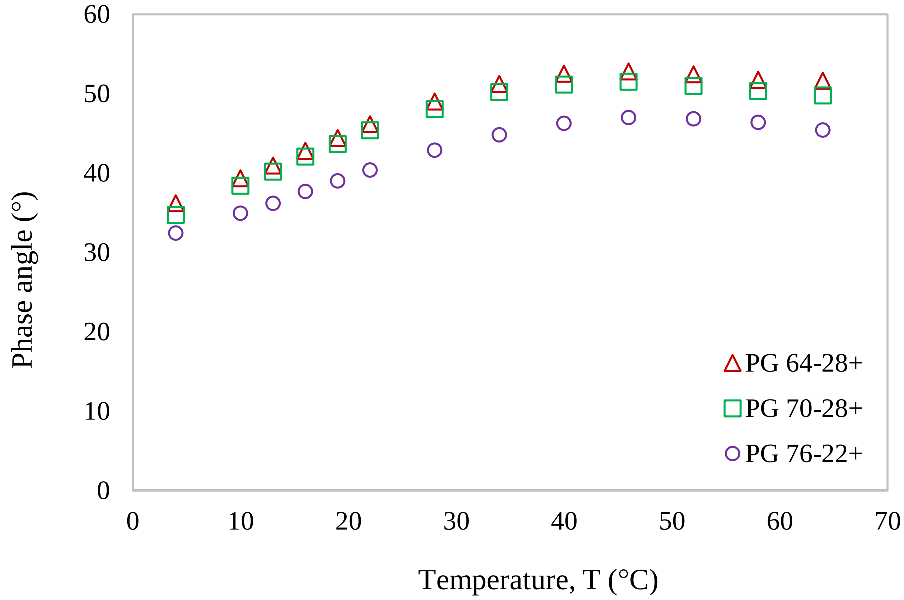


(b) Phase angle

FIGURE 19 DSR test results for RTFO-aged binders



(a) Complex modulus



(b) Phase angle

FIGURE 20 DSR test results for PAV-aged binders

PERFORMANCE-BASED CHARACTERIZATION

It is known that the rheological test cannot properly capture the performance of a polymer modified asphalt binder. Therefore, it is also recommended to apply different performance-based tests to evaluate the different damage performances of a binder. This study used multiple stress creep recovery (MSCR), and linear amplitude sweep (LAS) tests to determine the permanent deformation and fatigue damage performance under repeated loading. In addition, the elastic recovery test was done to verify the minimum percent recovery requirement specified by NMDOT.

Multiple stress creep recovery (MSCR) test

Test descriptions

The MSCR test determines the creep-recovery performance of an asphalt binder (*30*). In this test, the binder sample is subjected to a series of loadings using the DSR apparatus at two different stress levels. The stress levels in the MSCR test are chosen so that one stress is within the binder's linear viscoelastic (LVE) range, and another is just outside of it. This test method includes a small loading period to deform the specimen and a longer recovery period between loadings to assess the elastic recovery of the binder during the rest period. Figure 21 illustrates the stress and strain histories for one cycle.

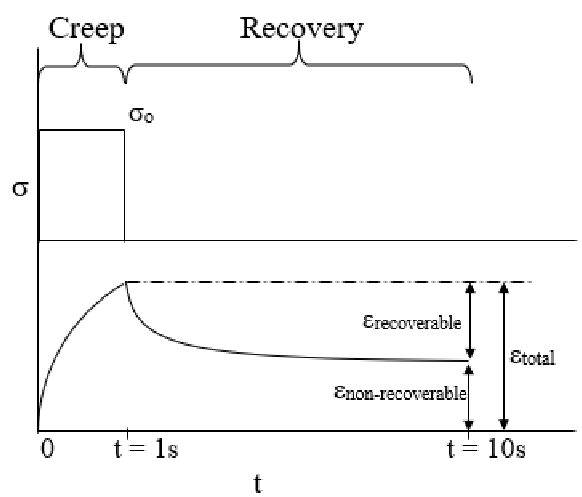


FIGURE 21 Strain history during Multiple stress creep recovery (MSCR) testing (*30*)

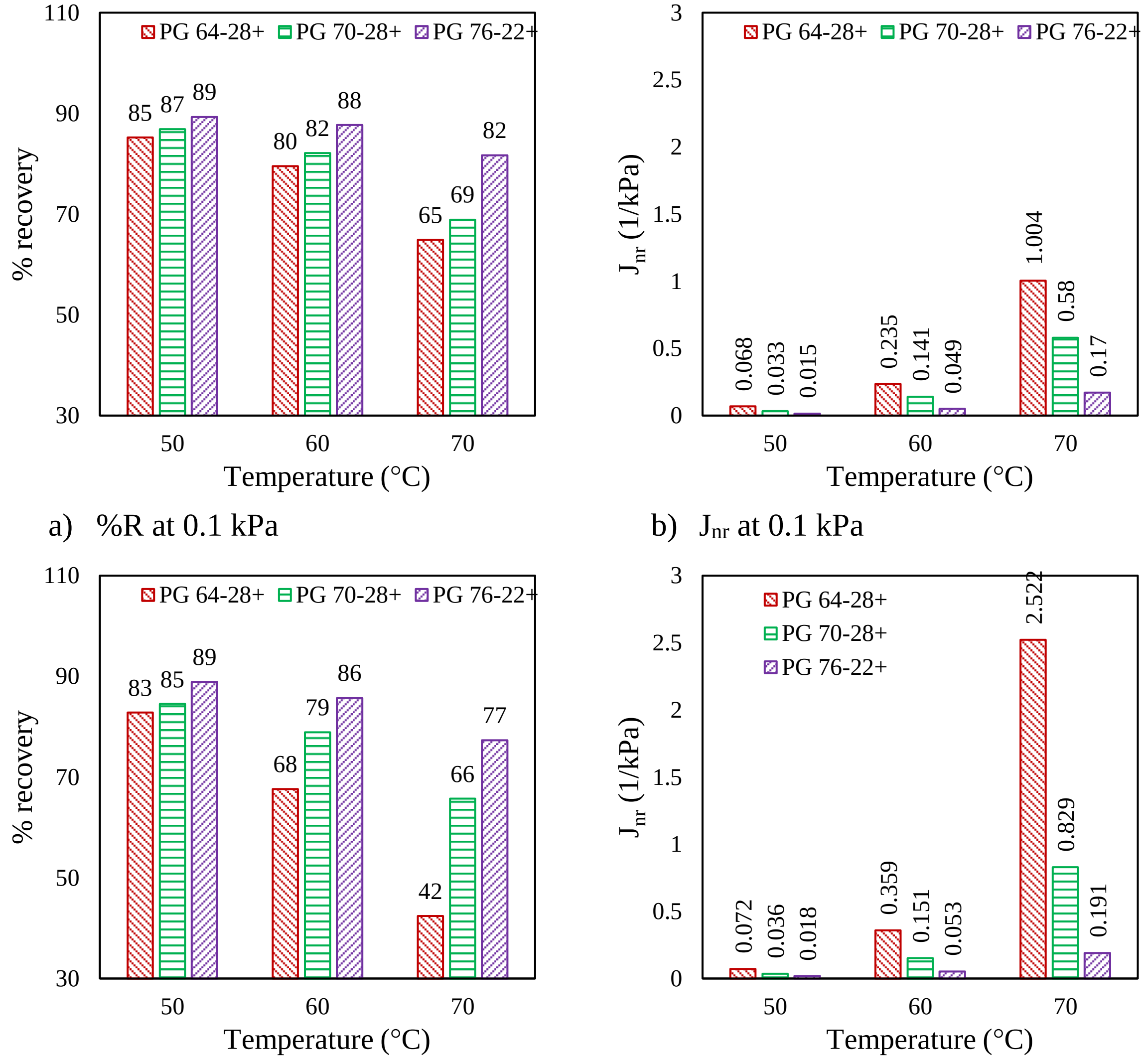
There are mainly two parameters that are being determined from the MSCR test: percentage recovery (%R) and non-recoverable creep compliance (J_{nr}). These parameters give an insight into the permanent damage performance of an asphalt binder. In this study, AASHTO T 350-14 was followed to conduct the MSCR test at three different temperatures (50, 64, and 70°C) on the RTFO aged binders. The %R and J_{nr} values at each cycle can be calculated using Eq. (6) and Eq. (7), respectively. The final %R and J_{nr} values of each stress level are the averages of all cycles at that respective stress level.

$$\%R = \frac{\varepsilon_{recoverable}}{\varepsilon_{total}} \cdot 100 \quad (\%) \tag{6}$$

$$J_{nr} = \frac{\varepsilon_{non-recoverable}}{\sigma_0} \qquad (1/\text{kPa}) \tag{7}$$

Test results

Figure 22 shows the MSCR test results for the RTFO aged binders. Figures 22(a) and 22(b) show the %R and J_{nr} values, respectively, for different temperatures at 0.1 kPa stress level. It is found that the %R increases as the high temperature grade of the binder increases. It is also evident that %R decreases with the increment of temperature. On the other hand, J_{nr} decreases as the high temperature grade of the binder increases and increases as temperature increases. Because PG 76-22+ binder is stiffer than the other two binders, the effect of temperature change on both parameters for this binder is lower than the other two binders. The effect of temperature change on both parameters for PG 64-28+ binder is the greatest among the test binders. Figures 22(c) and 2(d) show the %R and J_{nr} values, respectively, for different temperatures at 3.2 kPa stress level. Like 0.1 kPa stress level, binders have similar %R and J_{nr} rankings. However, %R at 3.2 kPa is lower than %R at 0.1 kPa for all cases. The degree of deterioration of %R due to increased stress is lower for PG 76-22+ binder and is greater for PG 64-28+ binder. An opposite trend has been observed for the J_{nr}. These results reveal that PG 64-28+ binder is more vulnerable to permanent damage than the other two binders and is also more temperature and stress sensitive. PG 76-22+ binder is stronger to withstand against permanent damage than the other two binders and is also less temperature and stress sensitive. The permanent damage performance of PG 70-28+ binder is slightly worse than PG 76-22+ binder and better than PG 64-28+ binder.



a) %R at 0.1 kPa
b) J_{nr} at 0.1 kPa
c) %R at 3.2 kPa
d) J_{nr} at 3.2 kPa

FIGURE 22 Multiple stress creep recovery (MSCR) test results

Linear amplitude sweep (LAS) test

Test descriptions

The LAS test is used to determine the fatigue damage performance of an asphalt binder under repeated loading (*31*). This test applies a limited number of incremental load cyclic loading on the sample to accelerate damage. This test uses the DSR apparatus and the standard 8 mm parallel plate geometry. The LAS test includes a frequency sweep test to obtain the response of the undamaged material and a strain sweep test at a constant frequency of 10 Hz to induce fatigue damage at an accelerated rate. Unlike the traditional 50% reduction method, the LAS test data needs to be analyzed by the simplified viscoelastic continuum damage (S-VECD) model to capture fatigue behavior. This model uses the viscoelastic corresponding principle with the work potential theory to derive the damage parameter (S) from the pseudo-stiffness (C) of a sample using Eq. (8).

$$S(t) = \sum_{i=1}^{N} \left[\frac{DMR}{2} \left(\gamma_p^R\right)^2 \left(C_{i-1} - C_i\right) \right]^{\alpha/_{1+\alpha}} \cdot \left[t_i - t_{i-1}\right]^{1/_{1+\alpha}} \tag{8}$$

where t is the time; N is the number of load cycles; α is equal to 1+1/m, and m is the slope of the logarithmic relationship between G^* and frequency (ω) that can be determined from a frequency sweep test; DMR is the dynamic modulus ratio = $G^*_{fingerprint} / G^*_{LVE}$, where $G^*_{fingerprint}$ is the initial G^* from the amplitude sweep test and G^*_{LVE} is the viscoelastic G^* from the frequency sweep test. The C can be calculated with Eq. (9).

$$C(S) = \frac{\tau_p}{\gamma_p^R \cdot DMR} \tag{9}$$

where τ_p is the peak stress in the cycle of interest, and γ_p^R is the peak pseudo-strain. γ_p^R can be calculated with Eq. (10) using the peak strain, γ_p in the cycle of interest, and an arbitrary modulus, G_R (assumed as 1).

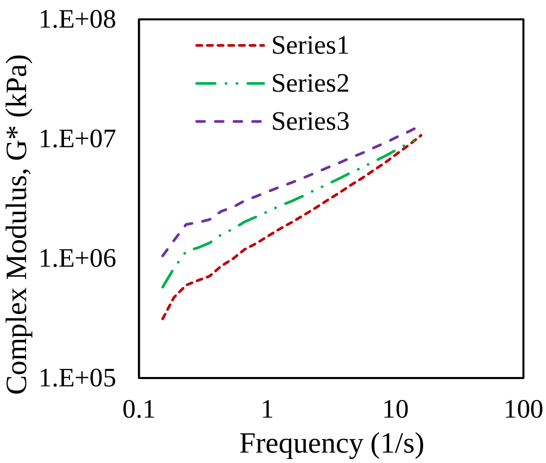
$$\gamma_p^R = \frac{1}{G_R} \cdot (\gamma_P \cdot G^*_{LVE}) \tag{10}$$

The relationship between C and S is referred as the damage characteristics curve of the material. For a given C, a higher S value means better resistance to fatigue damage and vice-versa. To define failure, stored pseudo strain energy, W_s^R needs to be calculated using Eq. (11). The number of cycles at which W_s^R reaches the peak value is assumed as the fatigue life.

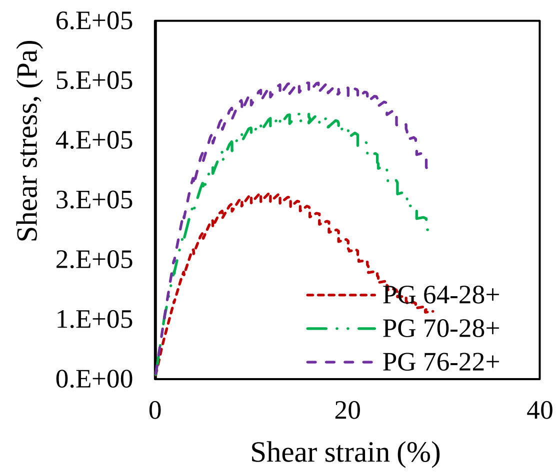
$$W_s^R = \frac{1}{2} \cdot C \cdot \left(\gamma_p^R\right)^2 \tag{11}$$

Test results

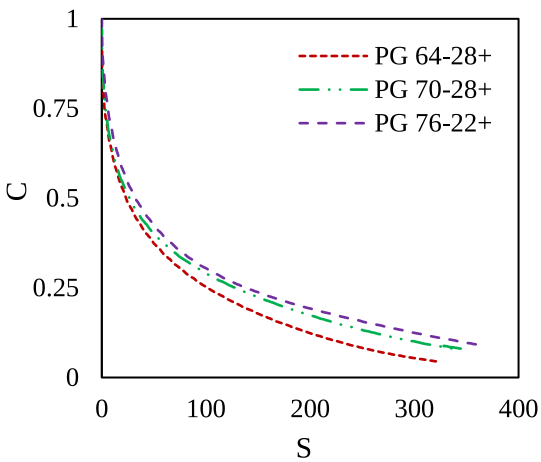
To evaluate the fatigue damage performance, the LAS test was conducted on PAV-aged binders. The test was done at 25 °C. Figure 23 compares the LAS test results for all three binders. Figure 23(a) shows the frequency sweep test results. It is found that the steady-state slopes of complex modulus in log space for the PG 64-28+, PG 70-28+, and PG 76-2+ binders are found as 0.786, 0.595, 0.487, and 0.359, respectively. The reason is that the stiffer binder (PG 76-22+) is less loading rate dependent than the softer binder (PG 64-28+). Figure 23(b) shows the effective stress-strain relationship for all tested binders. It can be seen that the peak stress increases as the high temperature grade increases. Figure 23(c) compares the damage characteristics curves of the tested binders. It shows that PG 76-22+ binder has favorable damage curve than the other two binders, and PG 64-28+ binder has the least favorable damage curve. Figure 23(d) shows the W_s^R values for all tested binders over the loading cycles. From this plot, the fatigue life is determined as the number of the cycle when the curve reaches the maximum value. For example, the fatigue life for PG 64-28+ binder is 1920 cycles. Figure 23(e) compares the respected fatigue lives of tested binders. The found fatigue lives of PG 64-28+, PG 70-28+, and PG 76-22+ binders are found as 1920, 2220, and 2420, respectively. These results demonstrate that PG 76-22+ binder has the best fatigue damage performance and PG 64-28+ binder has the worst fatigue damage performance



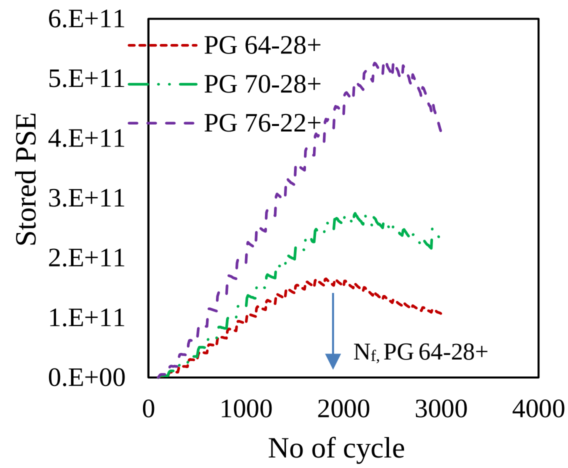
a) Frequency sweep test results



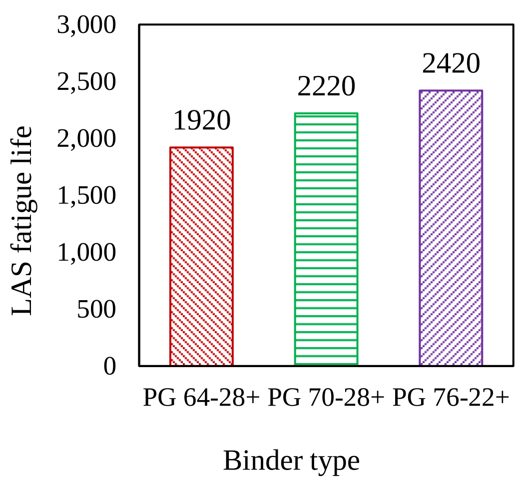
b) Amplitude sweep test result



c) Damage curve



d) Stored pseudo strain energy



e) Fatigue life

FIGURE 23 Linear amplitude sweep (LAS) test results

Elastic recovery test

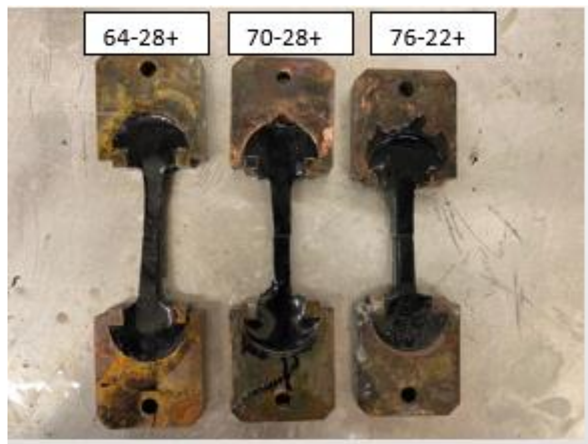
Test descriptions

The elastic recovery test is widely used by different DOTs to ensure a minimum percent recovery for the polymer modified binders. In this test, the binder sample is stretched up to 200 mm at a rate of 50 mm/min in the ductility meter. Then, the elongated sample is cut in the middle for recovery. After one hour of rest period, recovered length is measured in terms of initial length using Eq. (12). Figure 24 shows the elastic recovery test procedure used in this study.

$$\%recovery = \frac{(200 - x)}{200} * 100 \tag{12}$$

where x is the final reading in mm after bringing the two broken ends together.

a) Sample stretching



b) Test sample after rest period

FIGURE 24 Elastic recovery test

Test results

Table 10 presents the elastic recovery test results for all tested binders. It should be mentioned that the NMDOT specified %recovery value is a minimum of 65%. All tested binders passed the elastic recovery requirement. It is also found that elastic recovery increases slightly with binder grade.

TABLE 10 Elastic receovery test results

Binder type	% recovery
PG 64-28+	67%
PG 70-28+	70%
PG 76-22+	74%

SUMMARY

In this study, the rheological characteristics and performance of three different PG binders (PG 64-28+, PG 70-28+, and PG 76-22+) were investigated. It should be mentioned that all three binders are polymer modified binders. Based on the binder results of this study, PG 76-22+ binder has the most favorable rheological properties and permanent and fatigue damage performances among the tested binders. In contrast, PG 64-28+ binder is more susceptible to temperature and stress. As a result, it accumulates more damage during MSCR and LAS tests. Overall performance of PG 70-28+ is comparable to PG 76-22+ binder. The elastic recovery test results show that all binders passed the minimum elastic recovery value specified by NMDOT.

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LABORATORY PERFORMANCES OF OGFC

INTRODUCTION

This chapter focuses on laboratory performances of OGFC mixture specimens prepared with three proposed binders. To explore the effects of aggregate gradation on the OGFC's performance, three different aggregate gradations were also used in this study. Finally, laboratory compacted specimens were tested in the laboratory for various performance characteristics such as moisture susceptibility, abrasion durability (unaged and aged), and cracking performance. A new test was also developed to determine OGFC's durability performance in wet condition utilizing the Micro-Deval device.

SELECTION OF AGGREGATE GRADATIONS

In this study, three different aggregate gradations were selected. This study performed a cluster analysis to group the aggregate gradations used by different states.

Aggregate gradation clusters

Cluster analysis is a technique that can divide the data into groups based on similarities in the data (*32, 33*). In this study, aggregate gradations used by different states are grouped based on the average distance between the mid-point percent passing of each sieve for each gradation. As expected, three prominent clustered gradations were found from the analysis. These groups are separated based on their nominal maximum aggregate size. Cluster groups are shown in Figure 25.

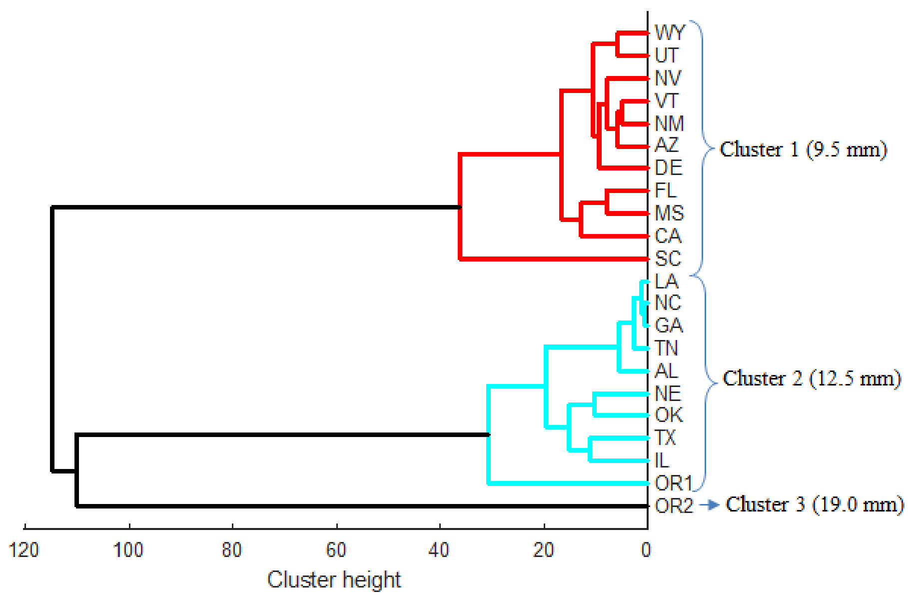


FIGURE 25 Aggregate gradation clusters

Selected aggregate gradations

Though it was planned to select one aggregate gradation from each cluster, this study did not use the 19.0 NMAS aggregate gradation (that is used in Oregon) because of its rarity. Instead, it used two 9.5 mm and one 12.5 mm NMAS aggregate gradations. Figure 26 shows three aggregate gradations used in this study. Between two 9.5 mm NMAS OGFCs, one (denoted as NMAS-9.5-mm-L) has lower fines than the other (denoted as NMAS-9.5-mm-H). The 12.5 mm NMAS OGFC is denoted as NMAS-12.5-mm.

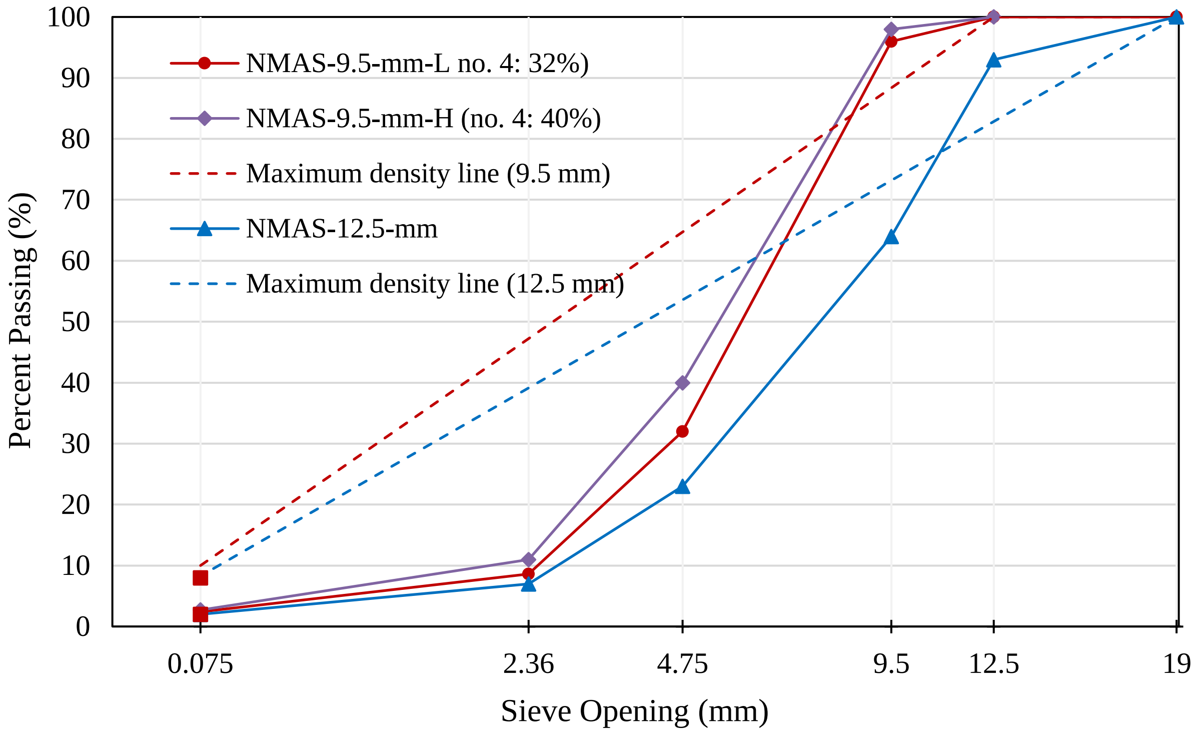


FIGURE 26 Aggregate gradations used in this study

MIX DESIGN OF OGFCS

The mix-design was done according to ASTM D7064 procedure. It is a variant of the NCAT OGFC mix-design procedure. Because NMDOT does not require durability (Cantabro abrasion) requirement, it was not considered during the mix design.

Determination of optimum binder content

The PG 70-28+ was used for the mix design purpose, and hence the determined binder contents were used for other binders. Hydrated lime was added as an anti-stripping additive at a rate of 1.0% by weight of aggregate. Figure 27 describes the mix design steps used in this study. In this study, aggregates were sieved first and then recombined according to their required proportions. Next, dry-rodded density and bulk specific gravity of coarse aggregate were determined. The

VCA_{DRC} (voids in coarse aggregate) was determined using the bulk specific gravity and dry-rodded density, as shown in Eq. (13).

a) Sieving of aggregate

b) Density test of aggregate

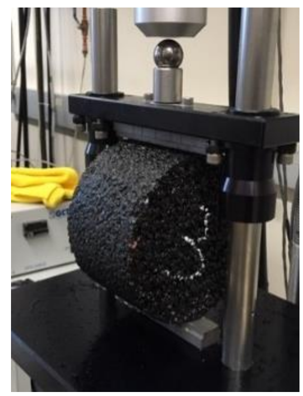
c) Bulk specific gravity test of aggregate

d) Laboratory mixing

e) G_{mm} test of loose mix

f) Drain-down test of loose mix

f) G_{mb} test of compacted specimen



g) TSR test

FIGURE 27 Aggregate gradations used in this study

$$VCA_{DRC} = \frac{G_{CA}\gamma_w - \gamma_s}{G_{CA}\gamma_w} x100 \tag{13}$$

where G_{CA} is the bulk specific gravity of the coarse aggregate, γ_w is the density of water, and γ_s is the dry-rodded density of coarse aggregate. After that, aggregate-asphalt binder blends were prepared with binder contents ranging from 6.0% to 7.5% (by weight of total mixture) at 0.5% increments. The prepared loose mix was used to determine the drain-down loss and theoretical maximum specific gravity, G_{mm}. The Core-lock method was used to determine G_{mm} of the loose mix. The drain-down test was done at two different temperatures: mixing temperature and 10° C above the mixing temperature. This was done to determine the potential loss while storing and transporting plant produced mix to the field. Next, the loose mix was compacted using the

Superpave gyratory compactor using 50 gyrations to determine air void contents, V_a and VCA_{MIX} from the bulk specific gravity, G_{mb} of compacted OGFC specimens using Eq. (14) and Eq. (15).

$$V_a = 100x(1 - \frac{G_{mb}}{G_{mm}}) \tag{14}$$

$$VCA_{MIX} = 100 - (\frac{G_{mb}}{G_{CA}} xP_{CA}) \tag{15}$$

where P_{CA} is the fraction of coarse aggregate, G_{mb} is the bulk specific gravity of compacted mix, and G_{mm} is the maximum theoretical specific gravity. There are three different criteria were used to determine the breakpoint sieve to calculate the percent of coarse aggregate, P_{CA} in a mix:

- Criterion 1: The breakpoint sieve is the no. 4 (4.75mm) sieve for all mixtures (*13*).
- Criterion 2: The breakpoint sieve is the sieve below which the slope of the gradation curve begins to flatten out (*34*).
- Criterion 3: The breakpoint sieve is the smallest sieve in which a minimum of 10% of the aggregate is retained (*34*).

This study found that the first criterion does not work for the used gradations and the last two criteria give similar results. The optimum binder content was determined as the binder content at which the air void, drain-down loss, and VCD_{MIX} values met the recommended criteria. Table 11 presents the mix design summary for NMAS-19.5-mm-L. Based on the mix design, the optimum binder content is determined as 6.5%. The mix design summary for NMAS-19.5-mm-H is shown in Table 12. The optimum binder content for NMAS-19.5-mm-H is similar to NMAS-19.5-mm-L (6.5%). Table 12 shows the mix design summary for NMAS-12.5-mm. The optimum binder content for NMAS-19.5-mm is 6.4%. Due to lack of fines, it was more prone to have a drain-down loss.

TABLE 11 Mix design summary for NMAS-9.5 mm-L

AC (%)	G_{mm}	G_{mb}	V_a (%)	VCA_{MIX}	VCA_{DRC}	DD loss %
6.0	2.380	1.918	19.4	30.9	42.3	0.13
6.5	2.372	1.942	18.1	30.0	42.3	0.17
7.0	2.363	1.962	17.0	29.3	42.3	0.59
7.5	2.341	1.941	15.9	29.1	42.3	0.72

TABLE 12 Mix design summary for NMAS-9.5 mm-H

AC (%)	G_{mm}	G_{mb}	V_a (%)	VCA_{MIX}	VCA_{DRC}	DD loss %
6.0	2.401	1.928	19.7	33.0	45.2	0.08
6.5	2.378	1.942	18.3	32.5	45.2	0.13
7.0	2.354	1.953	17.0	32.2	45.2	0.35
7.5	2.315	1.934	16.5	32.8	45.2	0.54

TABLE 13 Mix design summary for NMAS-12.5 mm

AC (%)	G_{mm}	G_{mb}	V_a (%)	VCA_{MIX}	VCA_{DRC}	DD loss %
6.0	2.361	1.901	19.5	30.8	44.6	0.17
6.5	2.344	1.917	18.2	30.2	44.6	0.31
7.0	2.325	1.922	17.3	30.0	44.6	0.59
7.5	2.308	1.934	16.2	29.6	44.6	0.73

PERFORMANCE CHARACTERIZATION

After determining the optimum binder contents, nine different OGFC mixtures were produced from three binders and three gradations. Next, different performance tests such as, drain-down loss, permeability, tensile strength ratio, Cantabro abrasion loss, Micro-Deval loss, and semi-circular bending tests were performed to evaluate the drain-down, moisture damage susceptibility, permeability, abrasion and aging resistance, durability at wet condition, and fracture performance, respectively.

Drain-down test

Test descriptions

It is recommended to perform the drain-down test at two different temperatures: the production temperature of the mixture and 10 °C above the production temperature. The drain-down test, as shown in Figure 28, is performed using a drain-down test basket that needs to be selected based on the aggregate size. In this study, the basket used has a sieve opening of 6.25 mm. Around 1,000 gm of hot loose OGFC mix (immediately produced from bucket mixing) is poured into the basket, and the initial weight of the sample is recorded. After that, a metal pan is placed under the basket to collect the segregated materials. The basket with the pan is then placed inside an oven for one hour (60 min) at the mixing temperature (and 10 °C above) of the mixture. After one hour, the weight of the segregated sample is measured. Finally, the percent loss is determined from segregated weight, M_s and initial weight, M_i, as shown in Eq. (16).

$$\%loss = \frac{M_s}{M_i} x100(\%) \quad (16)$$

a) Sample in test basket b) Basket in oven c) Weight of residuals

FIGURE 28 Drain-down test setup

Test results

In this study, the drain-down test was done on three replicate specimens to determine the drain-down loss of each OGFC. Table 14 shows the test results of NMAS-9.5-mm-L OGFC prepared with PG 70-28+ binder. It can be seen that the drain-down loss is more at a higher temperature than the production temperature. However, at both temperatures, the percent loss values are lower than the specified values.

TABLE 14 Drain-down test results for NMAS-12.5-mm-L prepared with PG70-28+

Test temperature (°C)	Trial no	Weight of sample placed in basket (gm)	Drain-down weight (gm)	% loss	Average % loss
Production temperature	1	1057.1	1.6	0.15	0.17
	2	1011.7	1.9	0.19	
	3	1208.4	2.1	0.17	
Production temperature + 10 °C	1	1101.2	3.2	0.29	0.26
	2	1086.4	2.7	0.25	
	2	1125.9	2.7	0.24	

Figure 29 compares the drain-down test results at their production temperatures among nine OGFCs. It shows that the drain-down loss varies by both binder grade and aggregate gradation. While comparing the binder grades, it can be seen that drain-down loss decreases as the binder grade increases. The binder with a lower grade (PG 64-28+) has lower stiffness than the binder with a high grade (e.g., PG 76-22+) and stiffness becomes further low at a higher temperature. Therefore, PG 64-28+ becomes more liquid at elevated temperatures to flow easily due to the gravitation force. While comparing the aggregate gradations, aggregate gradations with lower NMAS (9.5 mm) have lower drain-down losses than aggregate gradations with higher NMAS (12.5 mm). The reason is that fine aggregates have more surface area than coarse aggregates. In fine aggregates, the binder is more in coating form than a free stage. As a result, the binder cannot flow easily during the drain down test. With a further increment of fines, NMAS-9.5-mm-H has the lowest drain-down losses among the tested gradations.

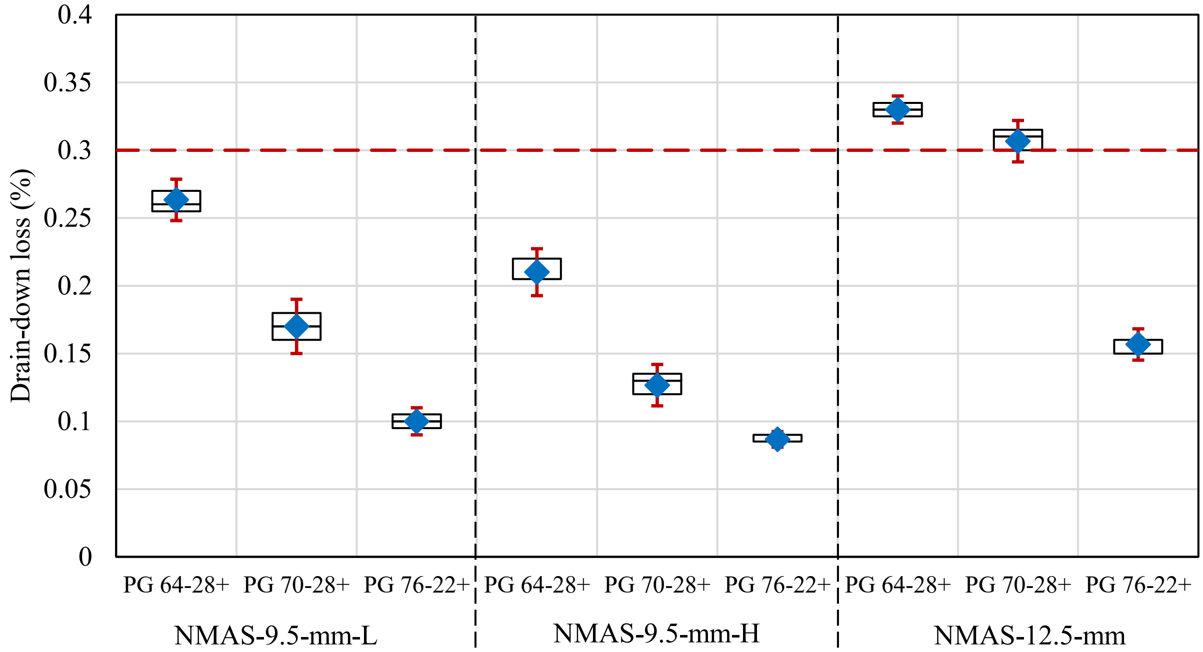


FIGURE 29 Drain-down test results

This study performed a *t*-test to see whether any OGFC fails to meet the NMDOT's drain-down requirement (maximum 0.3%). The *t*-test results are shown in Table 15. At 95% confidence level, NMAS-12.5-mm made with PG 64-28+ binder has the average drain-down loss, which is statistically greater than 0.3. Except for PG 64-28+ binder, the other binders pass the requirement in all cases.

TABLE 15 *t*-test results of drain-down performance

Parameters	NMAS-9.5-mm-L			NMAS-9.5-mm-L			NMAS-12.5-mm		
	PG 64-28+	PG 70-28+	PG 76-28+	PG 64-28+	PG 70-28+	PG 76-28+	PG 64-28+	PG 70-28+	PG 76-28+
n	3	3	3	3	3	3	3	3	3
df	2	2	2	2	2	2	2	2	2
Mean	0.26	0.17	0.10	0.21	0.13	0.09	0.33	0.31	0.16
std error	0.009	0.012	0.006	0.010	0.009	0.003	0.006	0.009	0.007
t-statistics	-4.16	-11.26	-34.64	-9.00	-19.65	-64.00	5.20	0.76	-21.50
Design value	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3
p-value	0.973	0.996	1.000	0.994	0.999	1.000	0.018	0.264	0.999
Significant	No	No	No	No	No	No	Yes	No	No

Tensile strength ratio (TSR) test

Test descriptions

The tensile strength ratio (TSR) test is used to determine the moisture damage susceptibility of the asphalt mixture. The TSR test is done by performing the indirect tensile (IDT) strength test on cylindrical asphalt concrete samples in both unaged and aged conditions. The unaged condition refers to the undamaged samples, whereas the aged condition refers to the moisture damaged samples after a moisture conditioning or freeze-thaw cycle. The moisture conditioning is done according to AASHTO T 283. The IDT test is conducted by loading a cylindrical specimen across its vertical diametral plane at 50 mm/min (2 in./min) deformation rate and 25 °C test temperature. The peak load at failure is recorded and used to calculate the Indirect Tensile (IDT) Strength of the specimen using Eq. (17).

$$IDT = \frac{2 * P}{\pi D t} \tag{17}$$

where *IDT* is the indirect tensile strength, *P* is the maximum compressive strength, *D* is the diameter of sample, *t* is the thickness of sample. Finally, the TSR value can be calculated from Eq. (18).

$$TSR = \frac{IDT_{aged}}{IDT_{unaged}} x100(\%) \tag{18}$$

Test results

In this study, three replicate specimens were used to determine the IDT strength at unaged and aged conditioning states. Figure 30 shows the specimen conditioning processes according to AASHTO T283. The test setup is known in Figure 31.

a) Saturating the sample b) Sample in freeze c) Sample in hot water

FIGURE 30 Specimen conditioning according to AASHTO T273

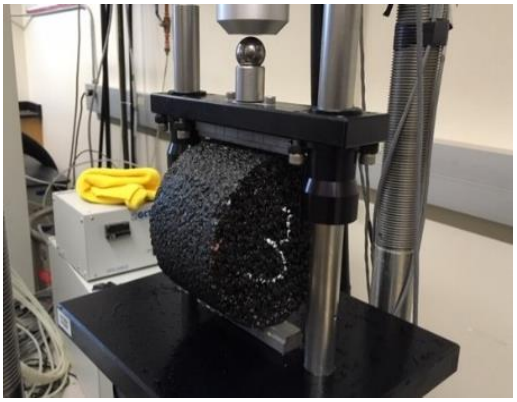


FIGURE 31 Indirect tensile (IDT) strength test setup

The IDT strengths of unaged specimens are presented in Figure 32. It can be observed that IDT strength increases as the binder grade increases regardless of aggregate gradation. From the binder test results, it is known that both stiffness and damage tolerance increase as the temperature grade of the binder increases. Thus, PG 76-22+ OGFCs have the highest IDT strengths, followed by PG 70-28+ OGFCs regardless of aggregate gradation. For the same reason, PG 64-28+ OGFCs have the lowest IDT strengths among the tested OGFCs. While comparing IDT results among the aggregate gradations, it can be seen that IDT strengths are higher for the OGFCs with lower NMAS (9.5 mm). The relatively fine aggregates in the NMAS-9.5-mm OGFCs are closely packed together, and as a result, they have improved load-carrying capacity than NMAS-12.5-mm OGFC. A similar trend is also observed for aged conditions (Figure 33). However, due to aging, every OGFC has a lower IDT strength than its unaged condition.

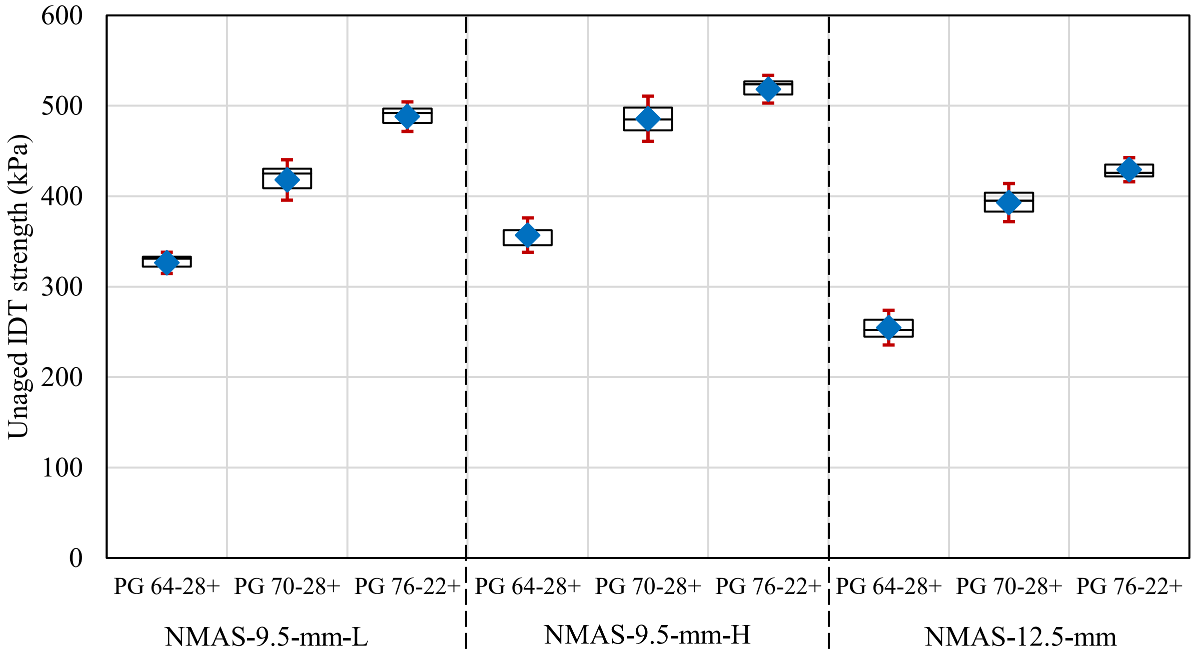


FIGURE 32 IDT test results for unaged specimens

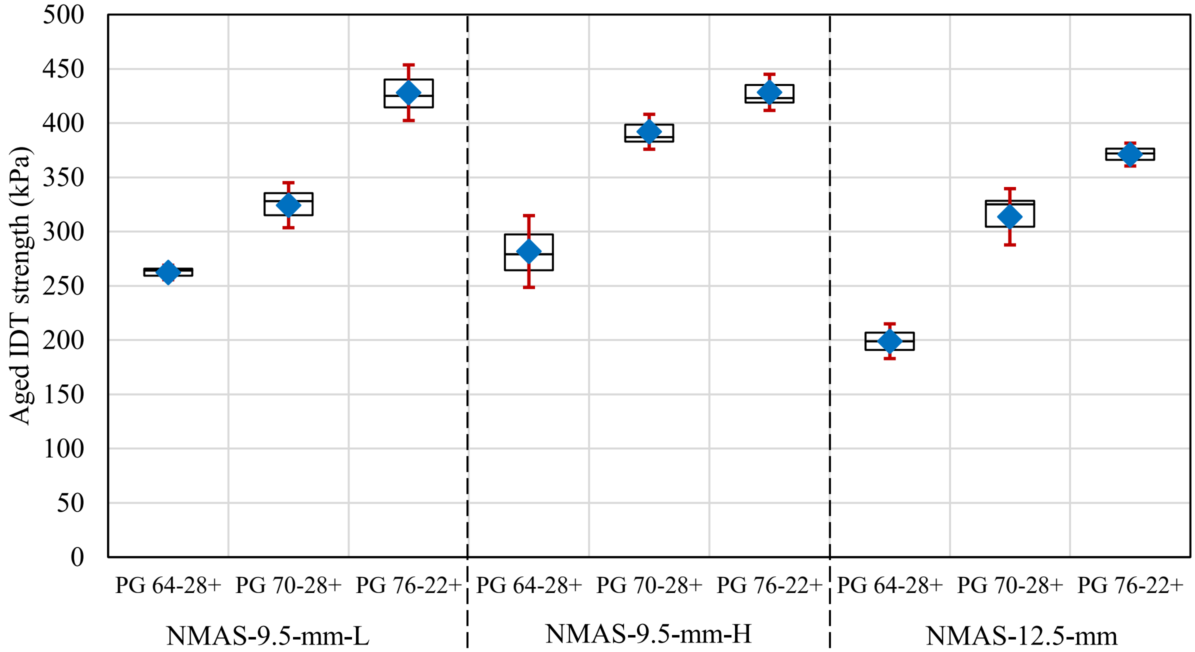


FIGURE 33 IDT test results for aged specimens

Figure 34 compares the TSR values among the nine test OGFCs. It shows that the TSR values of all tested OGFCs are around 80%. It indicates that all OGFCs meet the TSR requirement. It should be noted that NMDOT only uses the TSR value as a performance measure for OGFC materials. Using only the TSR value, it is difficult to distinguish the performance of different OGFCs. This indicates that a suitable performance test/parameter is required to properly characterize OGFC's performance.

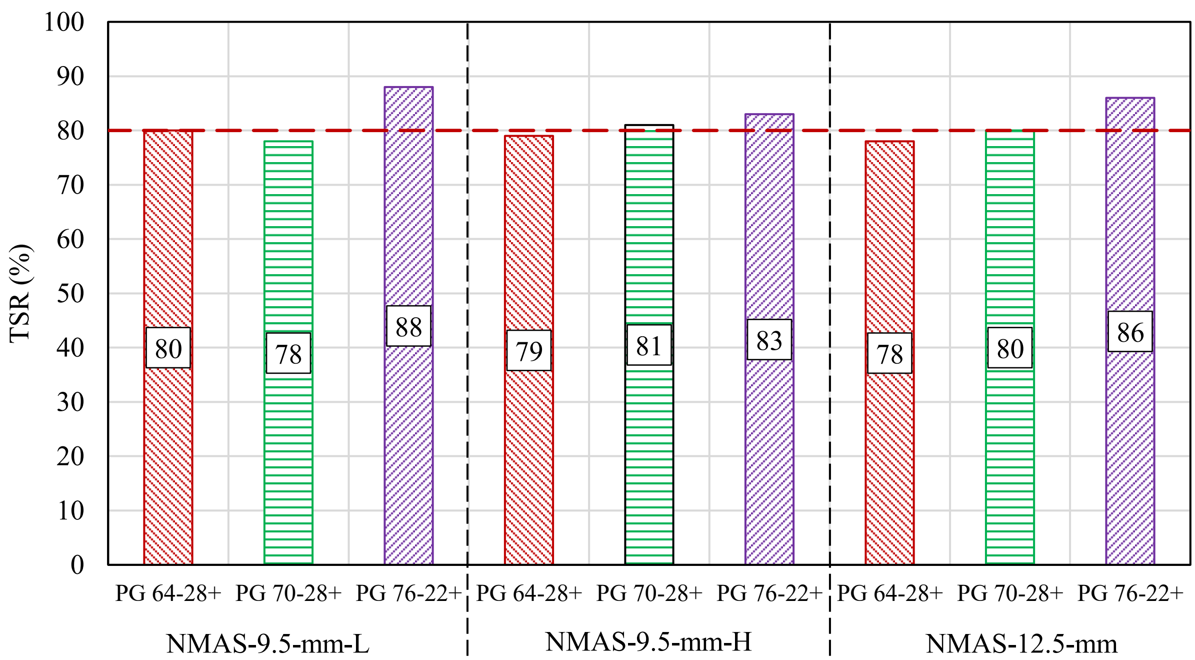


FIGURE 34 TSR values

Permeability test

Test descriptions

The permeability test method covers the laboratory determination of the water conductivity of the compacted asphalt paving mixture sample. A falling head permeability test apparatus (Figure 35) is used to determine the rate of flow of water through the specimen. Water in a graduated cylinder is allowed to flow through a saturated asphalt sample, and the interval of time taken to reach a known change in the head is recorded. The coefficient of permeability of the asphalt sample is then determined based on Darcy's Law, as shown in Eq. (19).

$$k = \frac{aL}{At} \ln\left(\frac{h_1}{h_2}\right) \tag{19}$$

where k is the coefficient of permeability, a is the cross-sectional area of the tube, A is cross-sectional, L is the thickness of the sample, t is elapsed time, h_1 and h_2 are the water heights during the test.

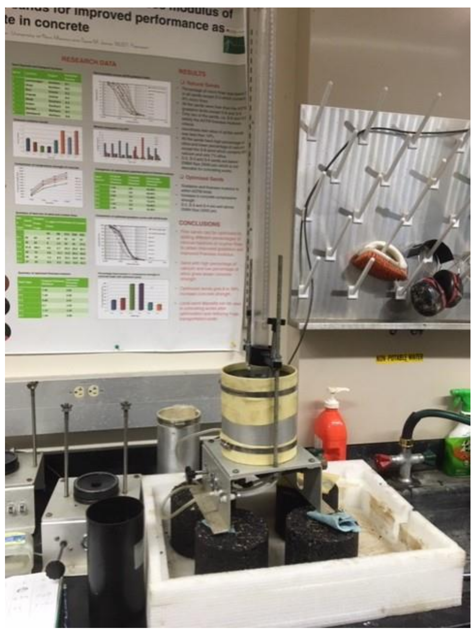


FIGURE 35 Permeability test setup

Test results

In this study, the permeability test was done on three replicate specimens. The test results are presented in Figure 36. It shows that binder grade has no effect on the permeability value. Permeability performance depends on the aggregate gradation of OGFC. NMAS-12.5-mm OGFCs have the highest permeability values regardless of binder type. Though the NMAS-9.5-mm OGFCs have the same air voids as NMAS-12.5-mm OGFC, the presence of more fine aggregates in the NMAS-9.5-mm OGFC clogged the interconnected pores and prevented the water from flowing. As a result, the permeability performance is lower for the NMAS-9.5-mm OGFCs. With little more fines, NMAS-9.5-mm-H has the lowest permeability between the two NMAS-9.5-mm OGFCs.

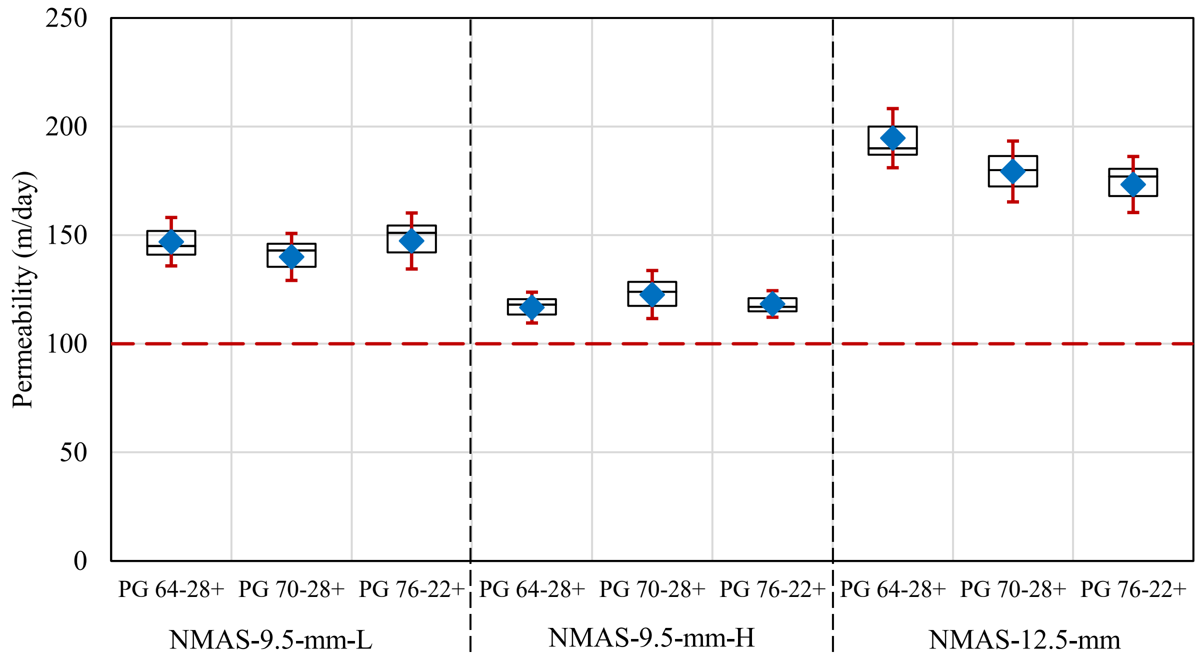


FIGURE 36 Permeability test results

This study performed a *t*-test to see whether any OGFC fails to meet the NMDOT's permeability performance requirement (minimum 100 m/day). The *t*-test results are shown in Table 16. At 95% confidence level, all other OGFCs pass the requirement.

TABLE 16 *t*-test results of permeability performance

Parameters	NMAS-9.5-mm-L			NMAS-9.5-mm-L			NMAS-12.5-mm		
	PG 64-28+	PG 70-28+	PG 76-28+	PG 64-28+	PG 70-28+	PG 76-28+	PG 64-28+	PG 70-28+	PG 76-28+
n	3	3	3	3	3	3	3	3	3
df	2	2	2	2	2	2	2	2	2
Mean	147.00	140.00	147.33	116.67	122.67	118.33	194.67	179.33	173.33
std error	6.429	6.245	7.446	4.096	6.386	3.528	7.860	8.090	7.446
t-statistics	-7.31	-6.41	-6.36	-4.07	-3.55	-5.20	-12.04	-9.81	-9.85
Design value	100	100	100	100	100	100	100	100	100
p-value	0.991	0.988	0.988	0.972	0.964	0.982	0.997	0.995	0.995
Significant	No	No	No	No	No	No	No	No	No

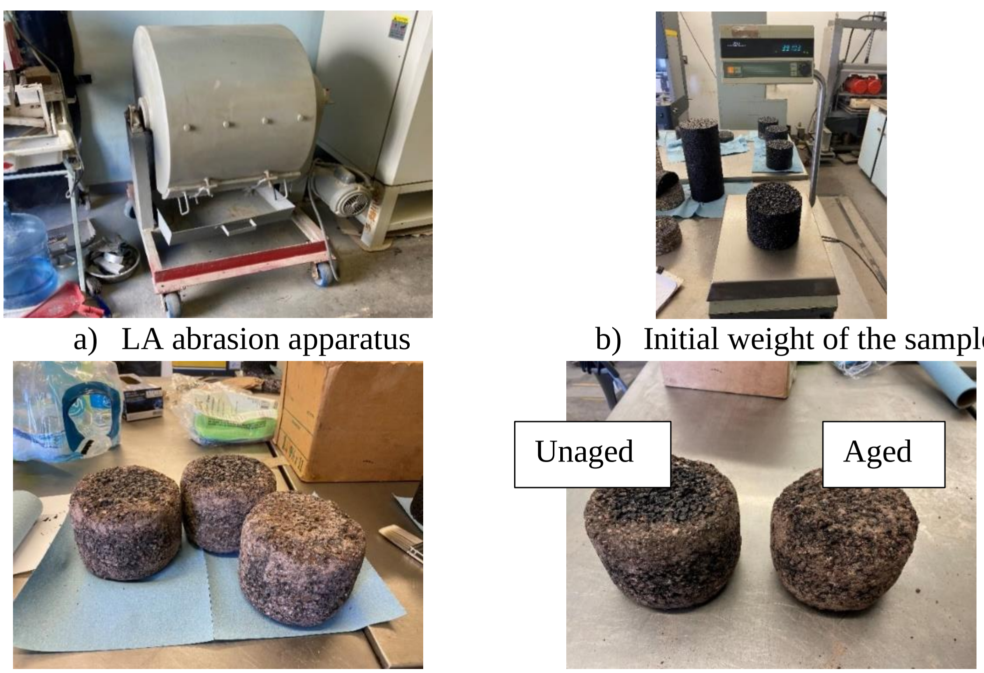
Cantabro abrasion (CA) test

Test descriptions

Since OGFC mixtures have relatively higher air void than the dense graded asphalt mixtures, many DOTs reported the durability problem for the OGFC materials. Therefore, several DOTs have been practicing a durability test for the OGFC mixtures, which is known as Cantabro abrasion (CA) test, as shown in Figure 37. Cooley et al. (*23*) provide a detailed literature review of the application of the CA test with OGFC. Developed in Spain in the 1980s, the CA test is one of the most widely used tests to evaluate the durability of OGFC mixtures. The test is done by using the Los Angeles (LA) abrasion drum for 300 revolutions at 30–33 rpm absent steel spheres at room temperature (generally at 25°C). The mass change due to abrasion is reported as the percent mass loss.

$$\%loss = \frac{M_1 - M_2}{M_1} x100(\%) \tag{20}$$

where M_1 and M_2 are the masses of the specimen before and after the test. The CA test is performed on both unaged and laboratory conditioned specimens to evaluate aging effects. The recommended percent mass loss is 15-20% for the unconditioned samples, whereas 25-30% for the moisture conditioned samples. The NCAT recommended values for the CA test are 20% and 30% for unaged and aged specimens, respectively. Several researchers evaluated the effectiveness of the CA test to assess OGFC durability performance, and they found that it is a good indicator of in-service OGFC durability performance (*35*).



a) LA abrasion apparatus

b) Initial weight of the sample

c) Broken specimen

d) Unaged vs. aged samples

FIGURE 37 Cantabro abrasion (CA) test setup

Test results

This study performed the CA test on three replicate specimens at each aging condition for each OGFC. The specimen height and diameter are 115 mm and 150 mm, respectively. The aged specimens were prepared by the AASHTO T283 procedure. The three replicate specimens of each OGFC were tested simultaneously. Thus, the percent loss was calculated using the total mass of the three specimens. The CA losses of unaged specimens are presented in Figure 38. It can be observed that the CA loss decreases as the binder grade increases regardless of aggregate gradation. The reason is that higher grade binders have higher stiffness as well as better damage tolerance. Thus, PG 76-22+ OGFCs have the lowest CA losses damage, followed by PG 70-28+ OGFCs regardless of aggregate gradation. For the same reason, PG 64-28+ OGFCs have the highest CA losses among the tested OGFCs. While comparing CA results among the aggregate gradations, it can be seen that CA losses are higher for the OGFCs with higher NMAS (12.5 mm). It is known that if fine particles are missing from the aggregate matrix, then the asphalt binder is only able to bind the coarse particles at their relatively few contact points. Thus, this kind of mixture has relatively lower stability than a mixture with more fines. The lack of fine aggregates in NMAS-12.5-mm OGFCs is the main reason for its poor performance. A similar trend is also observed for aged conditions (Figure 39). However, due to aging, every OGFC has more loss than its unaged condition.

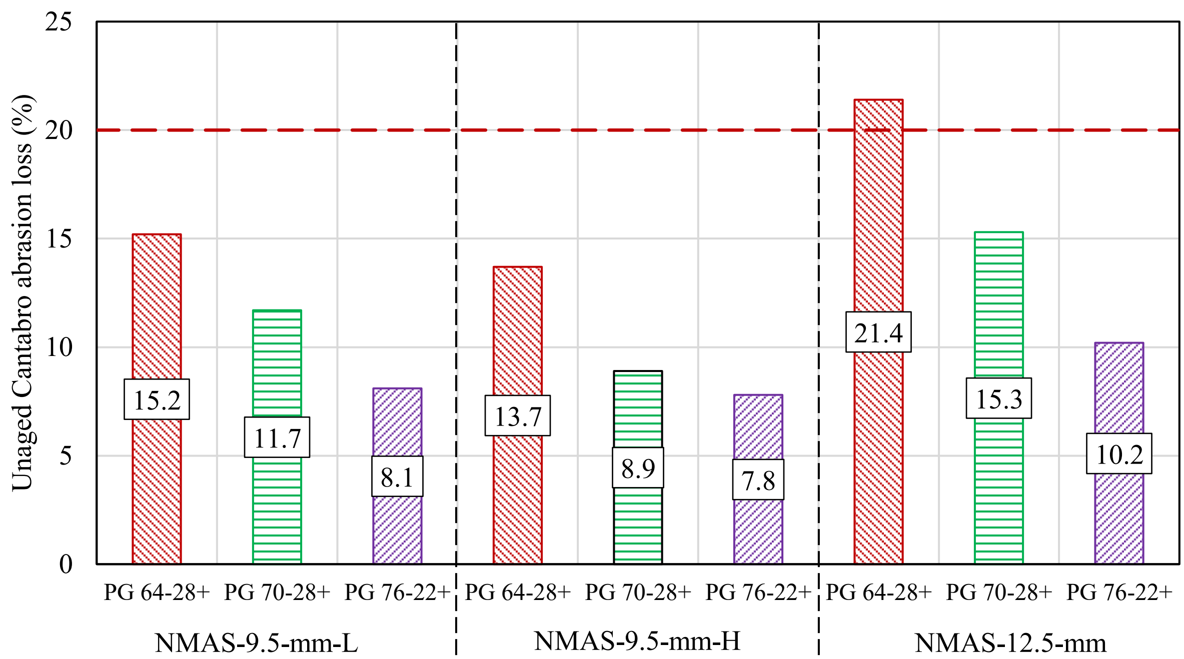


FIGURE 38 Cantabro abrasion (CA) test results for unaged specimens

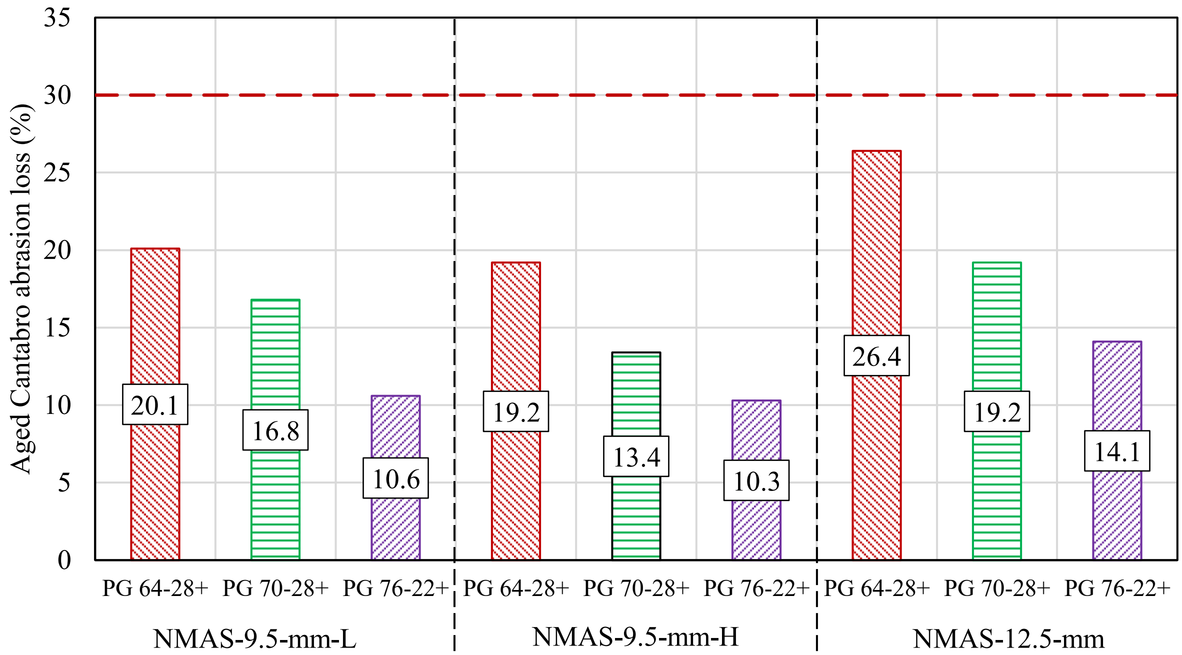


FIGURE 39 Cantabro abrasion (CA) test results for aged specimens

Micro-Deval loss test

Background

The current practice for determining the durability performance of an OGFC mixture is the CA test. The test is done only on the dry specimens (without the presence of water) in both cases. But in a practical scenario, it is very common that a wet OGFC layer is subject to the traffic loadings during/after the rains. Even after the rain, water can be clogged inside the voids. Under the wheel load, this entrapped water can create tremendous pressure on the void's wall and damage the pavement materials. Thus, the durability performance of OGFC material in wet conditions must be different than in dry conditions. However, currently, there is no such test available to measure the durability performance of an OGFC material in wet conditions. Therefore, this study evaluated the applicability of the Micro-Deval test (MDL) to determine the durability performance of an OGFC material in wet conditions.

Micro-Deval loss (MDL) test

The MDL test (as shown in Figure 40) is used to determine the abrasion resistance of aggregate as an alternative to the LA abrasion test. It has some advantages over the LA abrasion test. Besides the abrasion resistance, this test can also determine the durability of the aggregate (*36*). The MDL test data can be more spreadable than the LA abrasion test data (*37*). Unlike the LA abrasion test results, the MDL test results correlate better with the field performance (*38*, *39*). It is basically a wet LA abrasion test but using a smaller drum and smaller steel balls and in wet conditions. Kandhal and Parker (*40*) found that the MDL results correlate well to raveling, pop-outs, and potholing and suggested using the MDL test instead of the LA abrasion for this purpose. Several DOTs (such as Texas DOT, Oklahoma DOT, Utah DOT, and NCAT) adopted the MDL test to determine the abrasion and durability performance of their aggregates. The CA is performed using the presence of the steel charges. As a result, instead of measuring the loss due to the combination of abrasion, impact, and grinding, it can measure only the loss due to the abrasion. Therefore, the loading condition in the CA test is less representable to the field loading condition. Since the size of the steel ball for the MDL test is much smaller than the LA abrasion test, they can be included during the MDL test for making the loading condition more field representable. Furthermore, the MDL test is done in the presence of water, and the moisture damage susceptibility of the sample can also be determined. The UNM research team investigated the applicability of the MDL for the determination of the durability performance of OGFC mixtures in wet conditions.


a) Micro-Deval apparatus


b) Container and steel charges

FIGURE 40 Micro-Deval test setup (*41*)

Test descriptions

The test description is summarized in Figure 41. At first, cylindrical asphalt concrete samples were prepared in the lab. The height and diameter of the sample are 115 mm and 150 mm, respectively. As described before, the sizes of the drum/container of the MDL test are smaller than the LA abrasion test. Therefore, the sample needs to be trimmed with a 100-mm core so that it can be inserted into the test container, as shown in Figure 41(a). The wet samples are kept on a rack for at least 24 hrs to naturally dry out the moisture, as shown in Figure 41(b). After drying, the weight of each sample was measured (Figure 41(c)). The MDL apparatus is capable of performing two tests at a time. Therefore, two specimens were soaked in water at room temperature for 2 hrs, as shown in Figure 41(d). For the test, each wet specimen was inserted into one test container separately. Then, each container was filled with 2L water (at room temperature) and 5 kg steel charges, as shown in Figure 41(e). Next, each container was sealed with a cap and placed inside the MDL apparatus. The test was started by selecting the duration of the testing and pressing the start button. In this study, the duration of testing was chosen as 4 hrs, and the speed of the revolution was 100 rpm. After the completion of the test, the broken specimens were brought from the test container, washed thoroughly, and left for 24 hrs for natural drying. When the damaged samples were completely dry, the final weight was measured (Figure 41(f)). Finally, the percent loss was calculated using Eq. (21), where M_1 and M_2 are the masses of the specimen before and after the test.

$$\%loss = \frac{M_1 - M_2}{M_1} x100(\%) \tag{21}$$

a) Coring the specimen

b) Drying the specimen

c) Undamaged weight

d) Soak in water

e) Specimen and steel charges are in container

f) Damaged weight

FIGURE 41 Micro-Deval loss (MDL) test procedure (*41*)

In this study, three replicate specimens were investigated individually for each OGFC. Figure 42 shows the status of the damaged specimens after completion of the test. Figure 42(a) shows three replicate damaged specimens of NMAS-9.5-mm-L prepared with PG 64-28+ binder. It is clearly evident that all three specimens experienced sufficient damage during the test. It can also be seen that all three specimens are in similar damaged conditions. Because each specimen was tested individually, this observation confirms the repeatability of the MDL test. Figures 42(b) and 42(c) show the damaged specimens of NMAS-9.5-mm-L OGFCs prepared with PG 70-28+ and PG 76-22+ binders, respectively. In these two cases, the specimens got relatively less damage compared to the specimen prepared with PG 64-28+ binders. The primary reason is that PG 64-28+ binder is relatively less stiff and weak than the other two binders. Thus, it got more damage in the MDL test. Figure 42(d) compares the damaged specimens of three different binder grades. It can be seen that the damage ranking is based on their binder grades. This confirms that the MDL test can successfully rank the OGFC based on their performances.

(a) NMAS-9.5-mm-L (PG 64-28+)

(b) NMAS-9.5-mm-L (PG 70-28+)

(c) NMAS-9.5-mm-L (PG 76-22+)

(d) Effect of binder grade

FIGURE 42 Damaged samples after Micro-Deval loss (MDL) test

Test results

Figure 43 shows the MDL test results. Clearly, OGFCs made with PG 64-28+ binder have the highest losses than OGFCs made with the other two binders. During the test, heat generates due to the friction between steel charges and between steel charges and specimen. This increases inside water temperature, which makes the PG 64-28+ binder soft to accumulate damage easily. Among the aggregate gradations, NMAS-12.5-mm OGFCs have more damage than the other two OGFCs. It can also be seen that the standard deviations of all cases are very small. Thus, it can be concluded that the MDL test is repeatable.

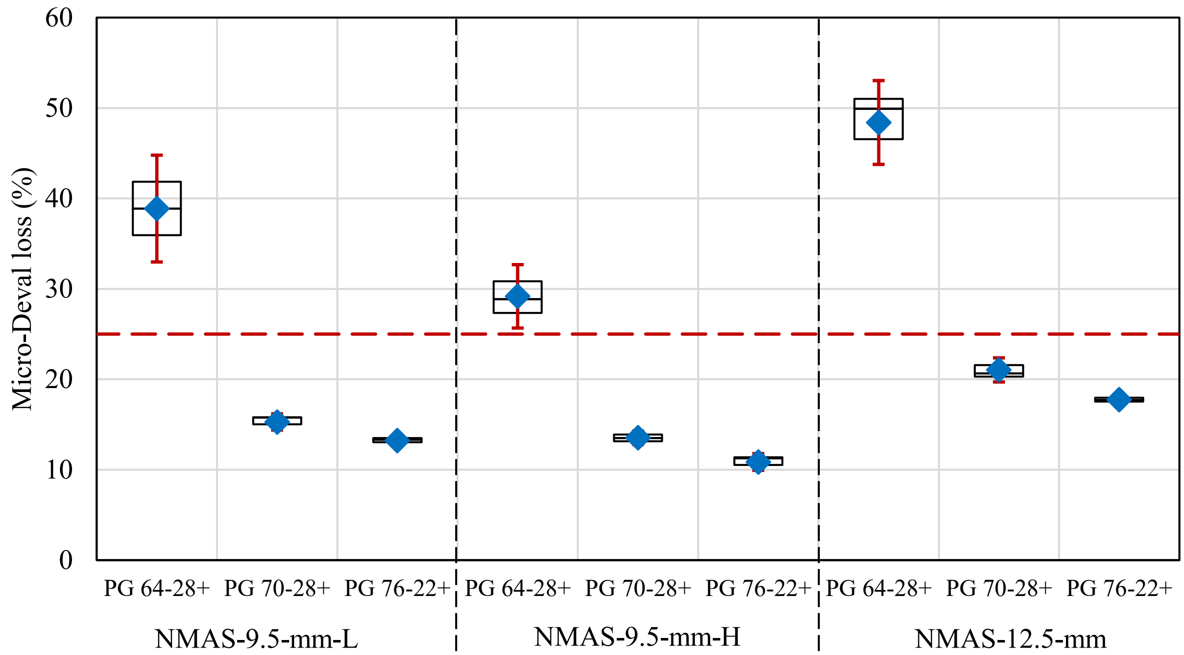


FIGURE 43 Micro-Deal loss (MDL) test results

Unlike the CA test results, the poor performance of OGFCs made with PG 64-28+ binder compared to OGFCs made with PG 70-28+ and PG 76-22+ binders can be clearly identifiable from the MDL test. To account for this poor performance, this study proposed a new criterion, a maximum 25% loss in the MDL test, to ensure adequate wet durability performance of OGFC. Table 17 shows the *t*-test results to see whether any OGFC fails to meet the requirement. It can be seen that all three OGFCs made with PG 64-28+ binder failed to meet the requirement.

TABLE 17 *t*-test results of Micro-Deal loss (MDL) performance

Parameters	NMAS-9.5-mm-L			NMAS-9.5-mm-L			NMAS-12.5-mm		
	PG 64-28+	PG 70-28+	PG 76-28+	PG 64-28+	PG 70-28+	PG 76-28+	PG 64-28+	PG 70-28+	PG 76-28+
n	3	3	3	3	3	3	3	3	3
df	2	2	2	2	2	2	2	2	2
Mean	38.88	15.29	13.24	29.17	13.54	10.86	48.40	21.04	17.77
std error	3.409	0.519	0.270	2.824	0.430	0.535	2.676	0.769	0.233
t-statistics	4.07	-18.73	-43.62	3.06	-26.65	-26.42	8.74	-5.15	-31.00
Design value	25	25	25	25	25	25	25	25	25
p-value	0.028	0.999	1.000	0.048	0.999	0.999	0.006	0.982	0.999
Significant	Yes	No	No	Yes	No	No	Yes	No	No

Semi-circular bending (SCB) test

Test descriptions

The semi-circular bending (SCB) test setup is shown in Figure 44. In this test, a pre-notched semi-circular specimen is loaded monotonically until fracture failure under a constant deformation rate of 0.5 mm/min and at a room temperature of 25 ^oC. During the test, the load and deformation are continuously recorded. Two parameters can be obtained from the test results, and they are strain energy to failure (*U)* and *J*-integral. The strain energy is defined as the work required to initiate a crack. The *J*-integral is known as the critical fracture energy release rate and represents the mixture's resistance to fracture. The higher the *J*-integral value, the mix is more resistant to fracture or crack propagation. It is determined using Eq. (22).

$$J-\text{integral} = -\frac{1}{b}\left(\frac{dU}{da}\right) \tag{22}$$

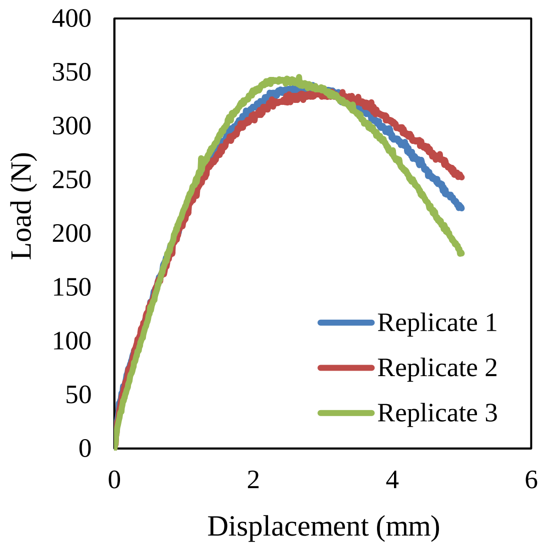
where *J*-integral is the critical strain energy release rate (J/m^2); *b* is the sample thickness (m); *a* is the notch depth (m); *U* is the strain energy to failure (J), and *dU/da* represents the change of strain energy with notch depth (J/m). In this study, the semi-circular specimens were prepared from cylindrical specimens compacted by the Superpave gyratory compactor. Three replicate specimens were tested at three different notch depth levels (15 mm, 25 mm, and 32 mm). The average thickness of the tested specimens is 60.0 mm.

(a) Sample (b) Test setup

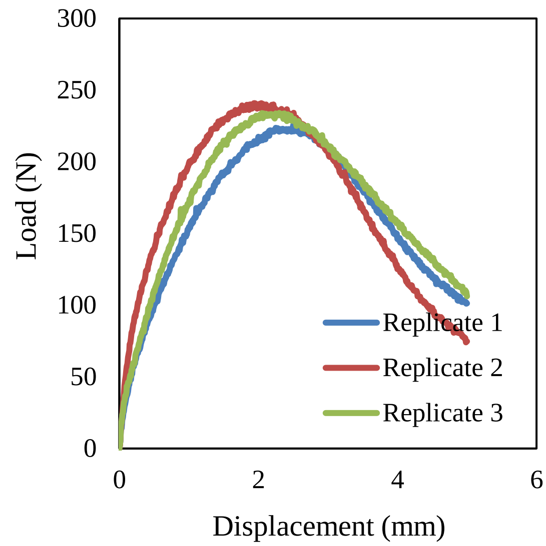
FIGURE 44 Semi-circular bending (SCB) test setup

Test results

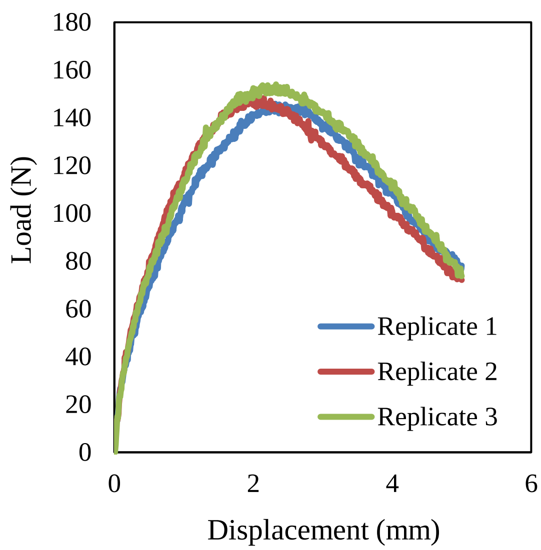
Figure 45 shows the SCB test result for the NMAS-9.5-mm-L prepared with PG 70-28+ binder. Figures 45(a)-(c) shows the load-displacement curves for different notch depths. It can be seen that the load-displacement curves for the three replicates are close to each other. This confirms the repeatability of test results. As expected, the peak failure load decreases with an increase in notch depth. It also found that the failure displacement corresponding to the peak load decreases with notch depth. Using these load-displacement plots, respective failure energies were calculated. The average failure energies for 15 mm, 25 mm, and 32 mm notched specimens are 0.65 J, 0.36 J, and 0.24 J, respectively, as shown in Figure 45(d). Later, these values are used to calculate the J-integral value of the tested mix, and the found value is 405 J/m^2.



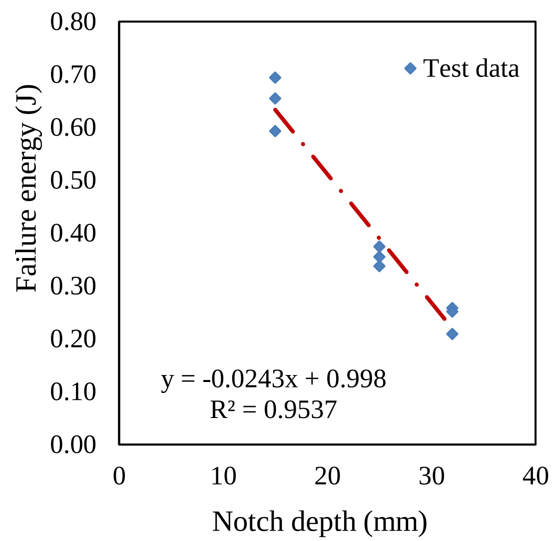
a) 15 mm notch



b) 25 mm notch



c) 32 mm notch



d) Failure energy versus notch depth

FIGURE 45 SCB test results for NMAS-9.5-mm-L prepared with PG 70-28+ binder

Table 18 summarizes the fracture energies of all tested specimens determined from the SCB test. It can be seen that the fracture energies of OGFCs prepared with PG 64-28+ binder are lower than the OGFCs prepared with the other two binders regardless of notch depth and aggregate gradation. This is because that PG 64-28+ binder is significantly weaker than the other two binders. Similarly, fracture energies of OGFCs prepared with PG 76-22+ binder are higher than the specimens prepared with the other two binders at 15- and 25-mm notch depths regardless of aggregate gradation. At 32 mm notch depth, fracture energies of OGFCs with PG 76-22+ and PG 70-28+ binders are comparable. From the table, it can also be observed that two NMAS-9.5-mm OGFCs have higher fracture strength than NMAS-12.5-mm OGFCs. Missing fines in NMAS-12.5-mm OGFC detriments the load carry mechanism and makes it more vulnerable to fracture. Between two NMAS-9.5-mm OGFCs, NMAS-9.5-mm-H performs slightly better than NMAS-9.5-mm-L. NMAS-9.5-mm-H has more fines than NMAS-9.5-mm-L, which makes it more stable and improves its loading carrying capacity.

TABLE 18 Semi-circular bending (SCB) test results for all OGFCs

Replicate no	Notch depth (mm)	NMAS-9.5-mm-L			NMAS-9.5-mm-H			NMAS-12.5-mm		
		PG 64-28+	PG 70-28+	PG 76-22+	PG 64-28+	PG 70-28+	PG 76-22+	PG 64-28+	PG 70-28+	PG 76-22+
1	15	0.272	0.655	0.670	0.313	0.735	0.811	0.130	0.488	0.529
2	15	0.241	0.694	0.660	0.357	0.858	0.942	0.159	0.461	0.503
3	15	0.215	0.593	0.697	0.319	0.919	1.055	0.149	0.392	0.507
1	25	0.171	0.375	0.416	0.209	0.427	0.525	0.096	0.294	0.304
2	25	0.166	0.337	0.364	0.176	0.433	0.470	0.084	0.313	0.329
3	25	0.181	0.355	0.381	0.195	0.525	0.458	0.093	0.267	0.296
1	32	0.054	0.258	0.209	0.112	0.297	0.308	0.064	0.208	0.173
2	32	0.063	0.209	0.206	0.134	0.282	0.317	0.068	0.198	0.170
3	32	0.079	0.251	0.181	0.138	0.316	0.276	0.087	0.154	0.194

Figure 46 shows the relationship between fracture energy and notch depth for NMAS-9.5-mm-L OGFCs prepared with all three binders. It can be observed that decreasing slopes of different binders are different. The OGFCs prepared with PG 76-22+ binder have the steepest slope. On the other hand, the OGFCs prepared with PG 64-28+ binder have the mildest slope. The slope for OGFCs prepared with PG 70-28+ binder is slightly mild compared to OGFCs prepared with PG 76-22+ binder. It is known that the slope of the notch versus fracture energy curve is related to the J-integral value, and a higher J-integral value represents that the mix is more resistant to fracture or crack propagation. Thus, it can be said that the OGFCs prepared with PG 76-22+ and PG 70-28+ binders are better cracking resistance than the OGFCs prepared with PG 64-28+ binder. Similar observations can be seen for other OGFCs (NMAS-9.5-mm-H and NMAS-12.5-mm), as shown in Figures 47 and 48.

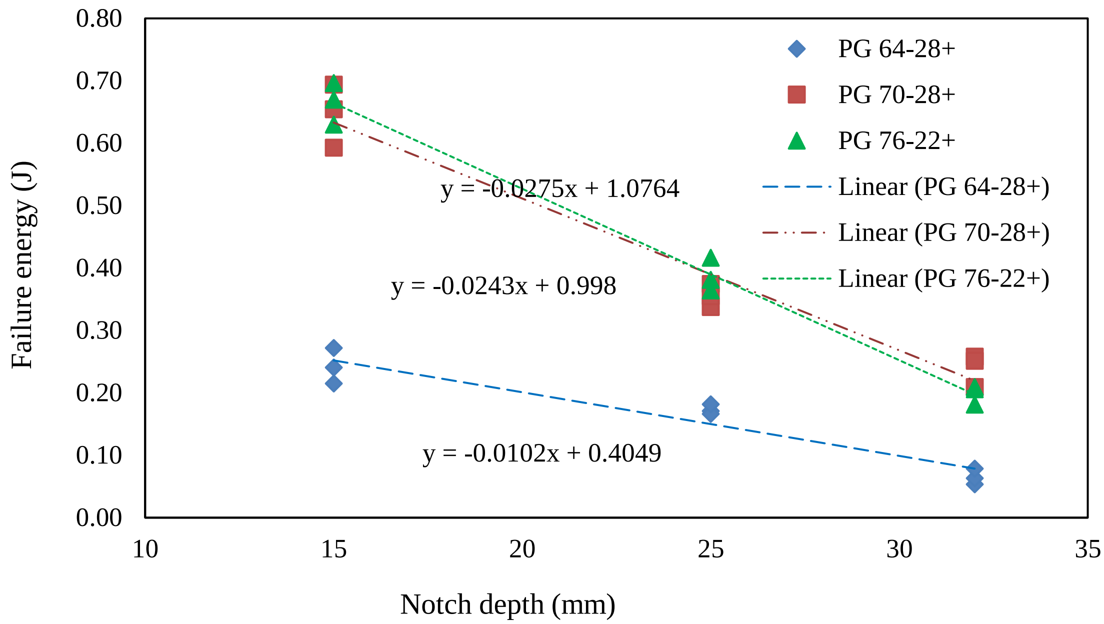


FIGURE 46 Notch depth vs. fracture energy for NMAS-9.5-mm-L

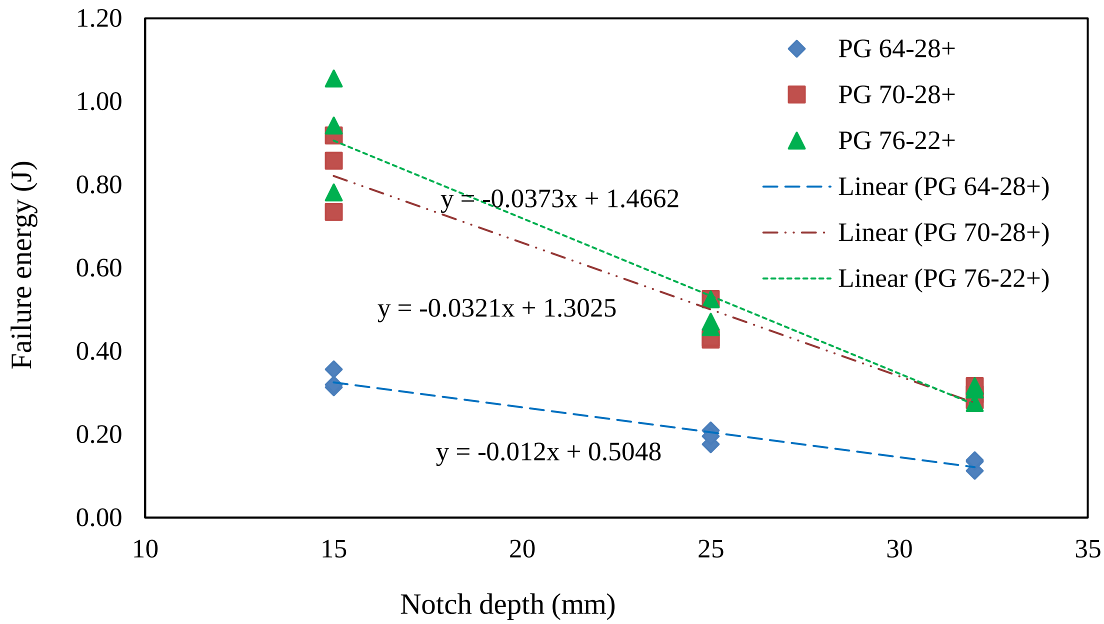


FIGURE 47 Notch depth vs. fracture energy for NMAS-9.5-mm-H

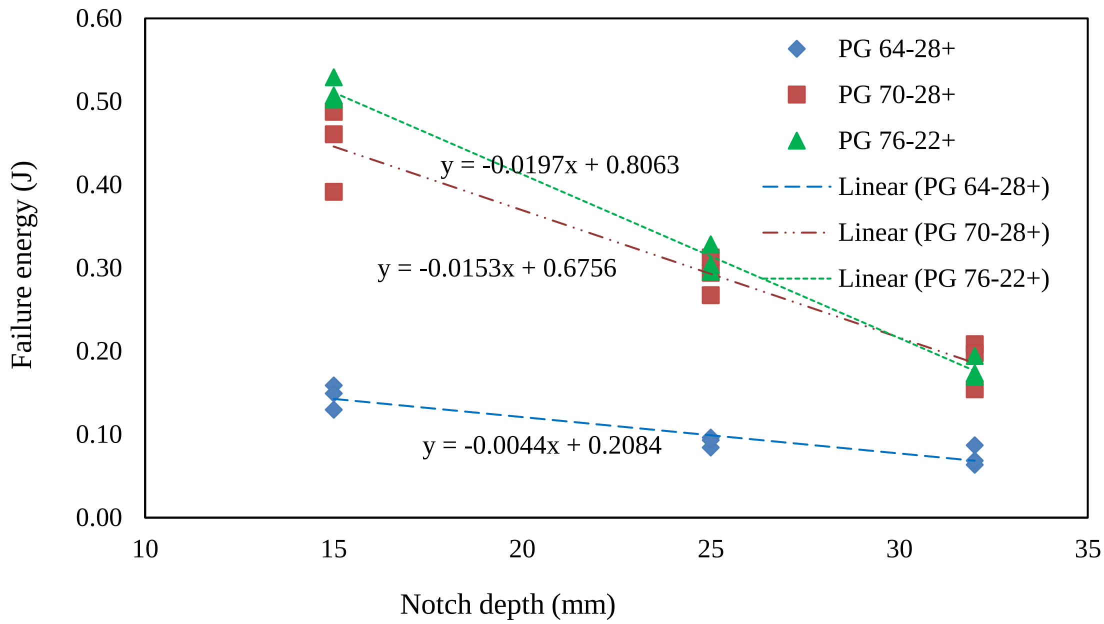


FIGURE 48 Notch depth vs. fracture energy for NMAS-12.5-mm

The computed *J*-integral values are listed in Table 19. It can be seen that the OGFCs prepared with PG 64-28+ binder have the lowest *J*-integral values regardless of aggregate gradation. Similarly, OGFCs prepared with PG 76-22+ binder have the highest J-integral values. OGFCs prepared with PG 70-28+ binder have slightly lower *J*-integral values than OGFCs prepared with PG 76-22+ binder. Among the aggregate gradations, NMAS-9.5-mm-H OGFCs have the highest *J*-integral values than the other two, while NMAS-12.5-mm OGFCs have the lowest values regardless of binder type. For a visual representation, *J*-integral values are also shown in Figure 49.

TABLE 19 *J*-integral from semi-circular bending (SCB) test

Binder type	*J*-integral (J/m^2)		
	NMAS-9.5-mm-L	NMAS-9.5-mm-H	NMAS-12.5-mm
PG 64-28+	170	200	74
PG 70-28+	405	535	255
PG 76-22+	460	620	328

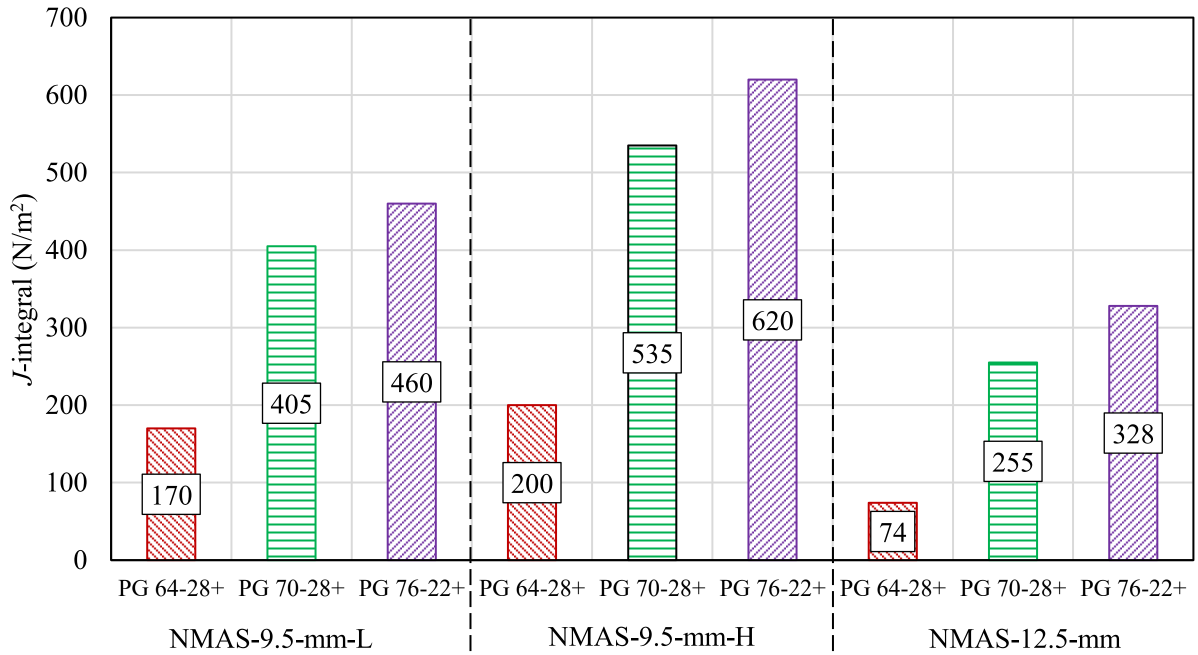


FIGURE 49 Comparison of *J*-integral values

SUMMARY AND PROPOSED BINDER GRADE ZONES

From the performance test results, it is observed that the volumetric specification cannot give any information about the performance of an OGFC. The performance test(s) is required to ensure the adequate performance of the OGFC mixture. In this study, all three binders met the volumetric requirements. However, it was clearly evident from the performance tests (e.g., IDT, CA, MDL, and SCB tests) that PG 64-28+ binder is weaker and performs poorly than the other two binders. Therefore, this study recommends only PG 70-28+ and PG 76-22+ binders for use in OGFC mixture. Since PG 76-22+ binder outperformed PG 70-28+ binder in all performance tests, this study also recommends using PG 76-22+ binder in the high demand pavements or in the hotter regions. As such, the revised binder grade zones for New Mexico are shown in Figure 50, which has only two zones.

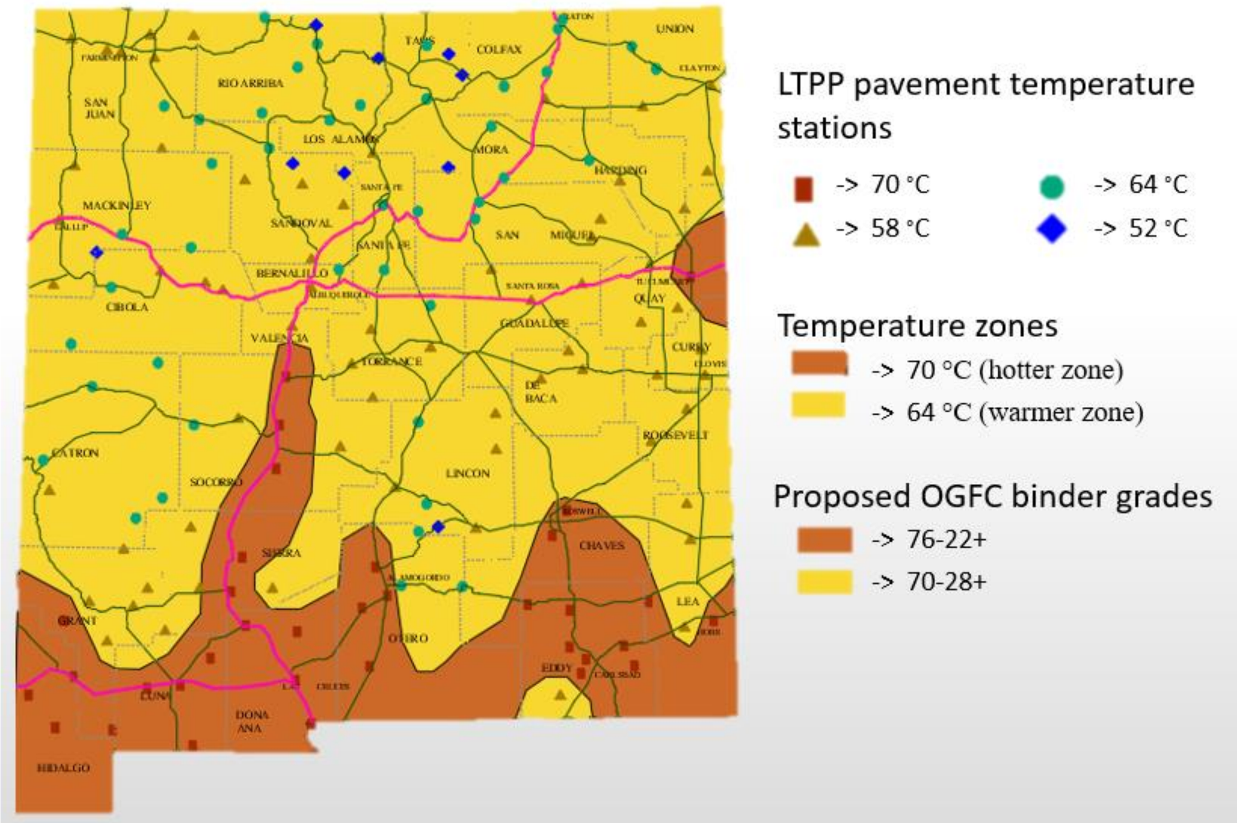


FIGURE 50 Final proposed binder grade zones for New Mexico

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CONCLUSIONS AND RECOMMENDATIONS

CONCLUSIONS

Currently, NMDOT uses only one performance grade binder: PG 70-28+ (or rubberized 70-28R+) all over the state, irrespective of climatic conditions. The primary objective of this study is to investigate whether they can use other binders (or binder grades) in their OGFC mixtures that provide equivalent or better performance. Another objective of this study is to evaluate the effectiveness of NMDOT's current volumetric mix design specifications based on only the TSR test. At first, a comprehensive literature review was performed to explore the current practices and past experiences of different states regarding OGFC materials. Next, this study compares the performances of two new binders with the current one to accomplish these project objectives. The new binders were selected considering the weather conditions of the state in such a way that one binder has one lower grade (PG 64-28+) and the other has one upper grade (PG 76-22+) than the current binder. This study also selected three different aggregate gradations (two 9.5 mm NMAS and one 12.5 mm NMAS) to evaluate the effect of aggregate gradation on OGFC's performance. Finally, a laboratory experimental program was designed to investigate the drain-down, moisture damage, abrasion, aging, wet durability, and fracture performances of nine OGFC mixtures prepared with these three binders and three aggregate gradations. Furthermore, binders' rheological properties and performances were also evaluated. Based on the study results, the following remarks can be made:

From the literature review:

- It is well accepted that OGFC improves safety and driving comfort. However, durability is a big issue for OGFC materials. Many colder states discontinued using OGFCs due to their poor durability performance and unfavorable winter maintenance requirements.
- Different state agencies use different aggregate gradations for their OGFC layers. The material selection, mix design, and performance evaluation practices also vary from state to state.
- PG 76-22 is the most common binder grade used in the pavement industry because it can perform better against drain-down segregation and provide a thicker coating that is required for OGFC. Many states recommend using fibers with binders to improve drain-down performance.

From the binder test results:

- From the binder tests, it is found that stiffness, temperature sensitivity, and damage performance improve as binder grade increases. PG 64-28+ binder has the lowest stiffness, highest temperature sensitivity, and worst damage performance among the tested binders, while PG 76-22+ binder has the opposite properties.

From the mixture test results:

- Drain-down test results show that the percent drain-down loss decreases as the binger grade increases because the softer binder flows easily due to the gravitational force at its higher (production) temperature. PG 64-28+ binder is susceptible to having more drain-down loss issues than both PG 70-28+ and PG 76-22+ binders because of its lower stiffness and higher temperature sensitivity. Among the tested binders, PG 76-22+ binder performs the best in the drain-down test.

- IDT test results reveal that the IDT strength increases as binder grade increases in both unaged and aged conditions. However, the TSR value is not affected by the binder grade. For example, PG 64-28+ OGFC has lower IDT strength than PG 70-28+ OGFC. However, the TSR values for both OGFCs are close to each other. PG 76-22+ OGFC has the highest IDT strength and a higher TSR value than other OGFCs.
- From both CA and MDL tests, it is found that PG 64-28+ OGFC got more damage than the other two OGFCs, and PG 76-22+ OGFC got less damage than PG 70-28+ OGFC.
- It is observed from the permeability test that the binder grade has no influence on the permeability performance of an OGFC.
- The SCB test results reveal that PG 64-28+ OGFC has the lowest fracture energy and *J*-integral than the other two OGFCs. PG 76-22+ OGFC slightly outperformed PG 70-28+ OGFC also in SCB test.
- It is observed from the permeability test that the binder grade has no influence on the permeability performance of an OGFC.
- The aggregate gradation affects the performance of an OGFC. For example, both NMAS-9.5-mm OGFCs performed better in IDT, CA, and MDL tests than NMAS-12.5-mm OGFC because NMAS-12.5-mm OGFC has fewer fine aggregates than NMAS-9.5-mm OGFCs. The missing of fine aggregates deteriorates its inter-aggregate load transferring mechanism and reduces overall load carrying capacity.
- Permeability performance is directly related to the aggregate gradation. NMAS-12.5-mm OGFC exhibited a higher permeability performance than both NMAS-9.5-mm OGFCs. Though all of them have the same air voids, relatively more fine aggregates in NMAS-9.5-mm OGFCs clogged the interconnect pores and reduced their permeability performances.
- Based on the tests' results, this study recommends using PG 76-22+ binder as an alternative to the current PG 70-28+ binder for better performance in the high traffic roads or in the hotter regions.
- This study also found that NMDOT's current volumetric mix design based on only the TSR test cannot ensure adequate durability performance of an OGFC mixture. For example, all three binders investigated in this study met the volumetrics and TSR requirements during mix design. However, PG 64-28+ OGFC performed worst in all performance tests.
- The CA and SCB tests can be useful to characterize abrasion, aging, and fracture performances of an OGFC mixture.
- The MDL test proposed in this study can be a good addition to other performance tests for evaluating the durability performance of an OGFC in wet conditions.

RECOMMENDATIONS FOR FUTURE STUDY

The following tasks can be recommended for the future studies:

- This is a preliminary study that investigated the performances of nine OGFC mixtures made with three different binders and three aggregate gradations. Adding more binders and gradations will make the conclusions more valid and acceptable.
- The conclusions presented in this study are based on only laboratory performances. However, the field pavement performances were not investigated here, which can be pursued in another research.

REFERENCES

1. Alvarez, A.E., Epps-Martin, A., Estakhri, C.K., Button, J.W., Glover, C.J., and Jung, S.H. Synthesis of Current Practice on the Design, Construction, and Maintenance of Porous Friction Courses, Report No. 0-5262-1, Texas Transportation Institute, Texas A&M University, College Station, Texas, 2006.
2. Transportation Research Circular, Open-Graded Friction Courses: State of the Practice, Number E-C005, Transportation Research Board, National Research Council, Washington D.C., 1998.
3. California Department of Transportation, Open Graded Friction Course Usage Guideline, MS # 5, Division of Engineering Services, Materials Engineering and Testing Services, Office of Flexible Pavement Materials, Sacramento, California, 2006.
4. Federal Highway Administration, Open Graded Friction course (OGFC) FHWA Mix Design Procedure, Report No. FHWA-RD-74-2, Federal Highway Administration, U.S. Department of Transportation, Washington D.C., 1974.
5. Federal Highway Administration. Open-Graded Friction Courses FHWA Mix Design Method. Technical Advisory T 5040.31. Federal Highway Administration, U.S. Department of Transportation, Washington D.C., 1990.
6. Kandhal, P.S. Design, Construction, and Maintenance of Open-Graded Asphalt Friction Courses. Information series 115. National Asphalt Pavement Association, Lanham, MD, 2002.
7. Watson, D., Johnson, A., and Jared, D. Georgia DOT's Progress in Open-Graded Friction Course Development. Transportation Research Record, No. 1616, 1998.
8. Cooley, L., Brumfield, J., Wogawer, R., Partl, M., Poulikakos, L., and Hicks, G. Construction and Maintenance Practices for Permeable Friction Courses. NCHRP Report 640, Washington D. C., 2009.
9. Nelson, J., Meta-Analysis of Airport Noise and Hedonic Property Values. Journal of Transport Economics and Policy, Vol. 38, No. 1, 1-27, 2004.
10. Shu X., Huang B., Song W., and Onyango M.A. Optimize Application of Open Graded Friction Course (OGFC) in Tennessee. 2017.
11. Yildirim, Y., Dossey, T., Fults, K., Tahmoressi, M., and Trevino, M. Winter Maintenance Issues Associated with New Generation of Open-Graded Friction Courses, Report FHWA/TX-08-0-4834-2, Center for Transportation Research, Austin, TX, 2007.
12. NCHRP Synthesis of Highway Practice 180, Performance Survey on Open-Graded Friction Course Mixes, Transportation Research Board, National Research Council, Washington D.C., 1992.
13. Kandhal, P.S. Design, Construction and Maintenance of Open Graded Asphalt Friction Courses. National Asphalt Pavement Association Information Series 115. May 2002.
14. Poulikakos, L., Takahashi, S., and Partl, M. A Comparison of Swiss and Japanese Porous Asphalt through Various Mechanical Tests. Conference paper Swiss Transport Research Conference 2004. 3rd STRC. Monte Verità/Ascona, 2003.
15. Tappeiner, W. Open-Graded Asphalt Friction Course. Information series 115. National Asphalt Pavement Association, Lanham, MD, 1993.

16. Khalid, H., and Pérez, F. Performance Assessment of Spanish and British Porous Asphalts. Performance and Durability of Bituminous Materials, Published by E & FN Spon, London, pp. 137-157, 1996.
17. Khalid, H., and Walsh, C. A Rational Mix Design Method for Porous Asphalt. In Procedures, 24th European Transport Forum, PTRC, Session G22i (Road Materials & Methods–Worldwide Update). London, 1996.
18. Fitts, G., The New and Improved Open Graded Friction Course Mixes. Asphalt, 17(2), 16, 2002.
19. Kandhal, P.S. Asphalt Pavements Mitigate Tire/Pavement Noise. Hot Mix Asphalt Technology, 22-3, 2004.
20. Newcomb, D., and Scofield, L. Quiet Pavements Raise the Roof in Europe. Hot Mix Asphalt Technology, pp. 22-28, 2004.
21. Jones, W. QUIET PAVEMENT — Coming to a Highway Near You. Asphalt, The Magazine of the Asphalt Institute, pp. 24-25, 2005.
22. Kearfott, P., Barrett, M., and Malina, J.F. Stormwater Quality Documentation of Roadside Shoulders Borrow Ditches. CRWR Online Report 05-02, Center for Research in Water Resources, The University of Texas at Austin, Austin, Texas, 2005.
23. NCHRP Synthesis of Highway Practice 284, Performance Characteristics of Open-Graded Friction Courses, Transportation Research Board, National Research Council, Washington D.C., 2000.
24. Mallick, R., Kandhal, P., Cooley, A., and Watson, D. Design, Construction and Performance of New Generation Open-Graded Friction Courses. National Center for Asphalt Technology Report 00-01. NCAT, Auburn, AL, 2000.
25. Danish Road Institute. Noise Reducing Pavements–State of the Art in Denmark. Report 141. DRI, Road Directorate, Ministry of Transport–Denmark, 2005.
26. Jackson, N. C., Vargas, A., and Pucinelli, J. Quieter Pavement Survey. Publication WA-RD 688.1. Washington State Transportation Commission, Department of Transportation, Olympia, WA, 2008.
27. Hasan, M.A., and Tarefder, R.A. Development of Temperature Zone Map for Mechanistic Empirical (ME) Pavement Design. International Journal of Pavement Research and Technology, 11(1), pp.99-111, 2018.
28. Hasan, M.A., Mannan, U.A., and Tarefder, R.A. Determination of Polymer Content in SBS Modified Asphalt Binder using FTIR Analysis. In International Conference on Construction in 19th Century, 2017.
29. Hasan, M.A., Khan, Z.H., Mannan, U.A., and Tarefder, R.A. Investigation of Mechanical Property of Polymer Modified Binder Using Image Processing and Finite Element Method. In International Conference on Transportation and Development 2018: Airfield and Highway Pavements. Reston, VA: American Society of Civil Engineers, pp. 227-234, 2018.
30. Hasan, M.A., Ahmad, M., Mannan, U.A., and Tarefder, R.A. Laboratory Investigation and Modeling of High Temperature Performance of Polymer Modified Binders (PMBs) with Different Concentration of Styrene-Butadiene-Styrene (SBS) Polymer. In Airfield and Highway Pavements 2019: Testing and Characterization of Pavement Materials. Reston, VA: American Society of Civil Engineers, pp. 248-254. 2019.

31. Hasan, M.A., Hasan, M.M., Bairgi, B.K., Mannan, U.A., and Tarefder, R.A. Utilizing Simplified Viscoelastic Continuum Damage Model to Characterize the Fatigue Behavior of Styrene-Butadiene-Styrene (SBS) Modified Binders. Construction and Building Materials, 200, pp.159-169, 2019.
32. Hasan, M.A., Islam, M.R., and Tarefder, R.A. Clustering Vehicle Class Distribution and Axle Load Spectra for Mechanistic-Empirical Predicting Pavement Performance. Journal of Transportation Engineering, 142(11), p.05016006, 2016.
33. Hasan, M.A., and Tarefder, R.A. Linking Traffic Volume and Weight Data for Mechanistic-Empirical Pavement Design. Journal of Transportation Engineering, Part B: Pavements, 146(2), p.04020015, 2020.
34. Watson, D.E., Masad, E., Moore, K.A., Williams, K., and Cooley, L.A. Verification of Voids in Coarse Aggregate Testing: Determining Stone-on-Stone Contact of Hot-Mix Asphalt Mixtures. Transportation Research Record No. 1891, 2004.
35. Alvarez, A. E., Epps-Martin, A., Estakhri, C., and Izzo, R. Evaluation of Durability Tests for Permeable Friction Course Mixtures, International Journal of Pavement Engineering, 11:1, 49-60, 2010.
36. Senior, S.A., and Rogers, C.A. Laboratory tests for predicting coarse aggregate performance in Ontario. Transportation Research Record, 1991.
37. Tarefder, R.A., Zaman, M., and Hobson, K. Micro-Deval Test for Evaluating Properties of Roadway Aggregate. International Journal of Pavements, 2(2). 2003.
38. Richard, J.A., and Scarlett, J.R. A Review and Evaluation of the Micro-Deval Test. Report ATR-024. Public Works and Government Services Canada, 1997.
39. Wu, Y., Parker, F., and Kandhal, P.S. Aggregate Toughness/Abrasion Resistance and Durability/Soundness Tests Related to Asphalt Concrete Performance in Pavements. Transportation Research Record, 1638(1), 1998: 85-93.
40. Kandhal, P.S., and Parker, F. Aggregate tests related to asphalt concrete performance in pavements. National Center for Asphalt Technology, 1997.
41. Hasan, M.A., Khan, Z.H., and Tarefder, R.A. Effects of Aggregate Gradations on the Performance of Open Graded Friction Course (OGFC) Mixtures. In International Conference on Transportation and Development, pp. 274-280, 2021.

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