Development of a 4.75 mm asphalt mixture design for Implementation in Louisiana DOTD Specifications

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Abstract

The Louisiana Department of Transportation and Development (DOTD) and other state agencies are continuously looking for techniques to reduce roadway maintenance and construction costs. A common consideration is to introduce asphalt mixtures with a smaller nominal maximum aggregate size (NMAS) for utilization in roadways. In a previous study, DOTD concluded that mixtures with a 4.75 mm NMAS provided acceptable performance as a surface layer. Excessive stockpiles of unused smaller aggregates can result in an economically competitive source to be considered for asphalt mixtures. The DOTD developed mixtures with four aggregate types and two binder types. A comprehensive evaluation of performance was conducted through volumetric and mechanistic testing. Performance testing consisted of the Loaded Wheel Tracking (LWT) test to determine rutting resistance, Semi-Circular Bend (SCB) test to evaluate intermediate temperature cracking resistance, and dynamic modulus (E^*) to ascertain the stiffness at intermediate temperatures. As expected, asphalt binder grade, aggregate type and mixture composition affected the performance of mixtures evaluated. Gravel mixtures were susceptible to cracking, while limestone mixtures were susceptible to rutting. An economic analysis was conducted to determine the viability of 4.75 mm mixtures. The cost per ton of 4.75 mm mixtures in Louisiana was higher than conventional 12.5 mm mixtures. However, when considering the lift thickness of potential overlays, the 4.75 mm aggregate mixtures became more viable. Further, a life-cycle analysis of a designed pavement using AASHTO Pavement-ME was performed to compare the lifetime durability of the 4.75 mm NMAS mixtures to a conventional 12.5 mm mixture.

Keywords: 4.75mm NMAS; Low aggregate size; Volumetrics; SCB; LWT; Mechanistic empirical design

1. Introduction and background

The Louisiana Department of Transportation and Development (DOTD) is facing budget constraints due to lack of funding. These challenges, combined with the fact that Federal Highway Administration (FHWA) annual funds have not received a significant increase in the past four years [1], have made it challenging to preserve the transportation infrastructure for the traveling public and industry in Louisiana. DOTD has focused research into techniques to reduce pavement maintenance and construction costs of asphalt pavements [2]. The use asphalt mixtures with smaller nominal maximum aggregate size (NMAS) may be a viable option to increase the number of miles a ton of mixture can pave.

In 2002, the National Center for Asphalt Technology (NCAT) completed a study to develop Superpave mix design criteria for a 4.75 mm NMAS mixture [3]. NCAT chose the permeability test (ASTM PS 121) to study different aggregate qualities that could enhance the performance of 4.75 mm NMAS mixtures.

Multiple federal and state agencies, such as AASHTO, Georgia, Maryland, and others, have included mixture design regulations for 4.75 mm NMAS mixtures in their specifications. As a result, several state highway agencies began using these recommendations of 4.75 mm NMAS mixtures for thin-lift applications, maintenance, and leveling courses to decrease construction time and provide an economical surface mix for low-volume roads [4].

Currently, the smallest NMAS mixture in DOTD's specifications is 9.5 mm (i.e., 3/8 inch) [5]. However, several studies [3,6–8] have established that, in specific scenarios, a lower NMAS mix (less than 12.5 mm) can perform better than mixtures that employ a larger NMAS. Louisiana DOTD is pursuing multiple goals by utilizing a lower NMAS mix; (1) decrease construction time, (2) provide an economical surface mix for low volume roads, (3) provide smooth riding surfaces, (4) thin-lift asphalt overlays, (5) correct surface defects, (6) leveling, (7) reduce permeability and (8) reduce the fine-aggregate stockpile for contractors.

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DOTD specifications has recently implemented balanced design techniques for asphalt mixtures. Therefore, asphalt mixtures are verified by specified rutting/moisture resistance and cracking criteria. The Hamburg Loaded Wheel Tracking Test (LWT) is used to evaluate the rutting and moisture resistance, while the Semi-Circular Bend Test (SCB) assesses intermediate temperature cracking resistance. In addition to these tests, this study conducted dynamic modulus (E^*) test in order to evaluate the stiffness of the 4.75 mm mixtures. E^* was used to predict the performance of the 4.75 mm mixtures via Pavement ME software. The performance prediction was utilized to perform a life-cycle cost analysis (LCCA). Cost analysis must consider the different overlay thicknesses used for different NMAS mixtures.

In order to thoroughly evaluate aggregate sources commonly used for roadway construction throughout the state, several mixture and aggregate designs were developed and subjected to mechanical testing.

2. Objective and scope

This study aims to evaluate the economic viability of the recently developed mixture design criteria for 4.75 mm NMAS mixtures to be used in Louisiana. This study utilized the data used to recommend the implementation of a 4.75 mm mixture design in DOTD specifications. The laboratory and mixture design data is included in the report. The cost analysis of the mixtures was combined with performance prediction using AASHTOWare Pavement-ME to generate the LCCA.

Commonly used aggregates and asphalt binders were evaluated to determine the most economical mixture for Louisiana DOTD. Five mixtures were developed for the testing factorial; these mixtures employed four aggregate types from variable sources. The aggregates were crushed gravel, 11 limestone, sandstone, and 910 limestone. Asphalt binder grades used for testing include conventional PG 67-22 and styrene-butadiene-styrene polymer modified PG 76-22m. These parameters were varied to understand the effect that aggregate and binder type have on 4.75 mm NMAS mixtures' performance. The cost and performance prediction of the five 4.75 mm mixtures were compared to conventional 12.5 mm and 9.5 mm mixtures.

3. Background

Currently, the lowest NMAS mixture allowed in Louisiana's specifications is 9.5 mm (3/8 inch) [3]. State agencies and research labs have shown smaller NMAS mixtures, specifically 4.75 mm, have several benefits over larger NMAS mixtures. These benefits include a reduction in screening stockpiles, use in thin lift applications, use in leveling and patching applications, and use in low volume road applications [6-8]. These benefits have made it a priority for state agencies, such as Louisiana DOTD, to evaluate the implementation of 4.75 mm mix designs into their standard specifications.

Using lower NMAS mixtures usually faces skepticism due to susceptibility to rutting. These rutting issues are caused by higher asphalt content in 4.75 mm mixtures compared to that of a standard 12.5 mm mixture. When the binder content is too high, it fills the void spaces and forces the aggregate particles to separate, which reduces the stone-to-stone contact; thus, rutting resistance is reduced [9]. In 2002, Cooley Jr. et al. [6] reported fine mixes have no more rutting potential than coarse mixes. In later studies, Williams [8] compared 4.75 mm mixtures to 12.5 mm mixtures using two-wheel tracker tests and confirmed the same phenomenon. Based on their studies, it was understood that 4.75 mm mixtures could be designed to resist rutting and stripping similar, and sometimes improved, when compared to 12.5 mm mixtures. Both studies found aggregate type, design air voids, and binder grade affected rut depths. Superpave specifications added a 4.75 mm NMAS designation and criteria section based on the work done by Cooley Jr. et al.

While aggregate selection depends on accessibility and cost, aggregate gradation is essential to a 4.75 mm mixture's performance and should follow the selected limitation [7]. Studies performed by Cooley Jr. et al. [3] and Zaniewski and Diaz [7] showed that limits of 30% to 54% passing the 1.18 mm sieve was reasonable. Table 1 shows the gradations from different agencies and research groups.

Dust content (percent of aggregate passing the 0.075 mm sieve) has a considerable effect on VMA and rutting. With an increase in dust content, VMA decreases, and vice versa. According to Williams [8], for every 3% increase in dust content, optimum binder content decreases by an average of 0.5%.

Table 1
Design criteria of state agencies and research of 4.75 mm NMAS mixtures.

Source	AASHTO [3]	Georgia	Maryland	Williams (2006) [8]	NCAT (2011) [10]
Gradations (percent passing)					
12.5 mm (1/2 in.)	95-100	90-100	100	100	95-100
9.5 mm (3/8 in.)	90-100	75 – 95	80-100	95-100	90-100
4.75 mm (No. 4)	-	60 – 65	36-76	90-100	-
2.36 mm (No. 8)	30-60	-	-	-	30-55
1.18 mm (No. 16)	-	-	-	30-60	-
0.3 mm (No. 50)	-	20 – 50	-	-	-
0.15 mm (No. 100)	-	-	-	-	-
0.075 mm (No. 200)	6.0-12	4.0-12	2.0-12	6.0-12	6.0-13
Design criteria					
Asphalt Content (%)	-	6.0-7.5	5.0-8.0	-	-
Air voids (%)	4.0	4.0-7.0	4.0	4.5 – 6.0	4.0
VMA (%)*	16 min.	-	-	18-20	-
VFA (%)**	75-78	50-80	-	71.9-75	-
DP (Percent Dust)	0.9-2.0	-	-	0.9-2.0	-

Rutting is also affected by Fine Aggregate Angularity (FAA) and natural sand. FAA and natural sand content need to be controlled in the mix to ensure a high degree of fine aggregate internal friction [7]. Cooley Jr. et al. [3] reports FAA should be 40% or greater for less than 0.3 million design ESALs and 43 and above for 0.3 to 3 million design ESALs. FAA criteria help to limit the rounded particles in the aggregate blend. Cooley Jr. et al. [3] suggests that natural sand should be limited to 15 - 20% for high volume roadways and 20 - 25% for low and medium volume roads. There is also evidence that natural sand content above 15% can adversely affect moisture and rutting susceptibility, as well as permeability [3]. Zaniwski and Diaz [7] found that over 10% of natural sand resulted in increased rutting, and over 20% of natural sand resulted in pronounced rutting potential. It is also agreed upon that too much natural sand can cause problems in the mix.

Design air voids have a significant impact on the mix. Williams [8] suggested air voids restricted between 4 - 6%. High volume roads typically use 4 - 4.5%, while low to medium volume roads can use 6% air voids due to less rutting potential. Higher air voids, such as 6%, help reduce binder content, which helps reduce the high costs of the binder. West et al. [10] also concluded that using a design air void range of 4% to 6% has little effect on the VMA. In the meantime, it will allow mix designers to reduce the asphalt content for a given aggregate blend when the VMA is well above 16.0%, which will improve the resistance of 4.75 mm mixtures to permanent deformation.

AASHTO has a minimum VMA requirement of 16% for 4.75 mm mixes, and it is the same for Superpave criteria. Williams [8] determined the critical VMA value from the relationship with dust content to be 16%, which matches AASHTO and Superpave criteria. Zaniwski and Diaz [7] suggest mixes designed 75 gyrations and above should have a maximum VMA of 18% to avoid excessive optimum binder. They state no maximum VMA criteria should be used for 50 gyrations mixes. If air voids are at 4% on a low volume road, a range of 16 - 18% for VMA may be used since low volume roads can tolerate higher values. If air voids and VMA are controlled, VFA is implied and not necessarily needed. The gradation and design criteria from different studies and state agencies of 4.75 mm NMAS mixtures are presented in Table 1 [11].

Historically, pavements were designed according to the Association of State Highway Officials (AASHTO), developed in the 1950s. This empirical method does not account for different climate conditions, detailed traffic information, and variability in materials properties. In order to overcome these limitations, a Mechanistic-Empirical (ME) design method was developed [12]. The Mechanical-Empirical pavement design has been adopted to help pavement engineers design pavement structures with more comprehensive knowledge. The pavement's mechanistic responses calculated based on the inputs of traffic, climate, materials properties and structures. Then, by utilizing the empirical response-distress relationship, the various distresses of pavement can be predicted. This approach can establish the fundamental relationship between materials properties and consequent distresses. This method can simulate pavement structure with more accurate and reliable inputs such as detailed climate and traffic data, material properties, and various construction procedures.

A life-cycle analysis of mixtures needs to be considered so that performance and durability can be compared to the costs of different mixtures with different components. Son and his team [13] considered cost-benefit analysis to define the advantages of 4.75-mm SMA mixtures. They have considered field and lab performances to compare the mixtures.

4. Methodology

4.1. Aggregate blends and mixture description

Aggregate types that are commonly used in Louisiana roads were selected for this study. Four aggregate types have been evaluated for this project: crushed gravel, 11 limestone, sandstone, and 910 limestone. Multiple aggregate blends were designed with one or two aggregate types, which satisfied the 4.75 mm NMAS aggregate criteria established by AASHTO. The four aggregate sources were combined with two different binder types resulting in eight mixtures for this study, described below. Table 2 presents the mixture naming convention used throughout this study.

4.1.1. 67-Gr and 76-Gr

Two mixtures utilized a gravel source aggregate. These mixtures used the same gradation of gravels but with different binder types. One used an unmodified PG 67-22 asphalt binder, and the other mix used a polymer-modified PG76-22m asphalt binder.

4.1.2. 67-Ls and 76-Ls

These mixtures were produced with the same limestone aggregate structure and blended with an conventional unmodified PG 67-22 and a polymer-modified PG76-22m asphalt binder.

4.1.3. 67-Gr+Ls and 76-Gr+Ls

These mixtures were developed to contain a combination of both gravel and limestone aggregates. The uniform aggregate design was blended with a base unmodified PG 67-22 asphalt cement binder and a polymer-modified PG76-22m binder.

4.1.4. 67-910 and 67-St

Two mixtures containing sandstone and 910 limestone were used for this project; they were blended with an unmodified PG67-22 asphalt binder.

4.1.5. 9.5 and 12.5 mm gravel

In order to have a comprehensive comparison, two conventional mixtures (9.5 and 12.5 mm NMAS) with gravel and unmodified PG67-22 asphalt binder have been produced.

4.2. Laboratory testing

Laboratory tests were conducted to determine the effectiveness of the aggregate designs, effect of binder type, and the various mixture types' overall performance. Laboratory evaluation methods utilized in this research are described below.

Table 2
Mixture types.

Mixture identification	Aggregate type	Binder PG grade
67-Gr	Gravel	67-22
76-Gr		76-22m
67-Ls	Limestone	67-22
76-Ls		76-22m
67-Gr+Ls	Gravel + Limestone	67-22
76-Gr+Ls		76-22m
67-910	910 Limestone	67-22
67-St	Sandstone	67-22
9.5 Gr	9.5 mm gravel	67-22
12.5 Gr	12.5 mm gravel	67-22

4.2.1. Ignition test

Ignition testing was conducted following with AASHTO T-308. A sample of approximately 1500 to 2000 grams of the mixture was gathered through quartering. The sample was transferred to an oven with a temperature of 530°C. The oven records sample's weight every minute, and the test will assume to be complete when the recorded weight remains constant. Simple calculations of the pre-ignition weight and post-ignition weight determine the asphalt weight. In addition to asphalt weight, the remaining aggregate is collected for gradation analysis based on AASHTO T-30.

4.2.2. Volumetric analysis

In order to have a comprehensive understanding of the various aspects of the 4.75 NMAS mix design, volumetric analysis was conducted. The volumetric analysis was used to determine the optimum asphalt content for all mixes. AASHTO M-323 was used to obtain the volumetric properties. Volumetric properties evaluated consisted of air voids (Va), voids in the mineral aggregate (VMA), voids filled with asphalt (VFA), and dust to asphalt proportion (DP).

4.2.3. Semi-circular bend (SCB) test

DOTD has implemented the SCB test to evaluate the cracking resistance through ASTM D8044-16. DOTD specifications require specific resistance to crack propagation in its roadway mixtures. This test is performed at 25 °C.

4.2.4. Hamburg loaded-wheel track (LWT) test

In order to determine the rutting resistance of the mixtures, the Hamburg LWT test was conducted according to the AASHTO T-324. DOTD specifications require a specific resistance to rutting. The rutting deformation was constantly recorded while a steel wheel with 703 N load passed over the submerged sample at a rate of 52 passes per minute. The water temperature was kept at 50°C water.

4.2.5. Dynamic modulus (E^*) test

This test was performed to evaluate the stiffness of the mixtures being subjected to cyclic loading. This test was conducted according to AASHTO T342.

4.3. Pavement performance evaluation using AASHTOWare Pavement-ME

In order to relate pavement structure and mixture properties such as layer thickness, modulus, and volumetric properties to the pavement response and eventually pavement performance, AASHTOWare Pavement-ME was employed to compare durability of smaller aggregate size mixtures to the conventional (12.5 mm NMAS) mixtures. This software requires a series of inputs to evaluate the pavement design including traffic information, climate data for the target region, local calibration factors for various distresses, and properties of materials used in the layers. Asphalt pavement performance is predicted for major distresses such as international roughness index (IRI), top-down and bottom-up fatigue cracking, asphalt layer rutting, total rutting, and thermal cracking. A 95% reliability level was considered for all distress types of having a relatively conservative design. New asphalt pavement was analyzed with a design life of 20 years, considering the following inputs. Based on previous research in Louisiana, the Louisiana pavement mechanistic-empirical design's

local calibration coefficients are implemented following the proposed guideline [14]. It has been shown that the AASHTOWare Pavement ME national-calibrated distress/IRI models generally underpredict bottom-up fatigue cracking and overpredicts rutting for flexible pavements.

4.3.1. Traffic inputs

A traffic level with an average annual daily truck traffic (AADTT) of 4000 was assumed. Although 4.75 mm NMAS mixture was considered for low traffic cases, a high traffic level was assumed as the worst-case scenario. In this research, AASHTOWare Pavement-ME default traffic was used. Based on the developed truck axle load spectra in the software, the normalized axle load distribution for single, tandem, tridem, and quad-axle type for vehicle classes 4 through 13 were used. A monthly distribution factor of 1 was considered. In terms of axle configuration, 2.6 m was used for average axle width. Also, 1.3 m, 1.2 m, and 1.2 m were used as tandem, tridem, and quad-axle spacing, respectively. Further, mean wheel location was considered 0.46 m, and traffic wander standard deviation was assumed as 0.25 m.

4.3.2. Materials and pavement structure

A total of three aggregate structures with NMAS of 4.75 mm, 9.5 mm, and 12.5 mm were selected for evaluation, Fig. 1. An asphalt layer with a thickness of 19.0 mm was used for 4.75 mm mix. Asphalt layers for 9.5 mm and 12.5 mm mixes were assumed to be 38.0 mm and 50.8 mm, respectively. A level 1 input for the dynamic modulus and binder performance grade of 4.75 mm and 12.5 mm mixes was used. A level 3 dynamic modulus input for the 9.5 mix was considered. A 203.2 mm stabilized base layer with a resilient modulus of 551.6 MPa was assumed. Also, a 254.0 mm non-stabilized layer with a resilient modulus of 186.1 MPa was considered for the sub-base layer. Subgrade was assumed to be a semi-infinite clayey layer with 110.3 MPa resilient modulus.

5. Discussion of results

A series of laboratory tests were performed following the objective of this research. These tests were designed in order to evaluate the performance of the 4.75 mm NMAS mixtures with multiple different aggregate combinations and compare them with 9.5 and 12.5 NMAS mixtures. These tests compared mixtures' behaviour through mixture design volumetric properties, LWT performance, and SCB performance. Results of the laboratory testing are presented in the following sections. Details of the mixture design development of the mixture evaluated in this study are provided in previous work [15].

Fig. 1. Pavement structures for (a) 4.75 mm mixture, (b) 9.50 mm mixture, (c) 12.5 mm mixture.

5.1. Mixture design

Table 3 presents the mixture designs evaluated in this study. Five aggregate structures were developed for this evaluation. Three of the aggregate structures were mixed with two asphalt binder grades, resulting in eight mixtures for assessment.

5.2. Semi-circular bend (SCB) test

Cracking sensitivity was determined using Semi-Circular Bending test (SCB) at Intermediate temperature [16]. The critical strain energy (J_c) of the mixtures is presented in Fig. 2. A desirable cracking resistance determined through the higher J_c value, while the state of Louisiana specified the criteria for acceptable cracking resistance (J_c) to be higher or equal to 0.5 kJ/m², for low volume roadways and greater than or equal to 0.6 kJ/m² for high volume roadways. All the mixes except the 67-22 Gravel passed the threshold of a minimum J_c of 0.50 kJ/m². Therefore, the mixtures developed in this study should not have concerns regarding premature intermediate temperature cracking. This result is consistent with small NMA mixture observations. Higher asphalt contents result in increased durability, as long as permanent deformation is not present.

It is noted that the use of polymer modified binder, consistently increased J_c values evaluated in this study. Again, this finding is expected due to the increased durability observed by using SBS polymer modification [17–18]. The gravel was the most susceptible to cracking, while limestone and limestone plus gravel performed similarly. Moreover, 910s and sandstone had the most cracking resistance, according to SCB J_c .

5.3. Loaded wheel-track (LWT) test

Permanent deformation is a significant concern with mixtures containing small aggregates, therefore, LWT was selected to characterize the behavior of the mixtures in response to cyclic rolling loads. The LWT results are shown in Fig. 3. The LWT results obtained presents the rut depth for each mixture. The DOTD specifications for Thin Lift asphalt mixtures (Section 501) requires dense mixture rut depth to be less than 12mm at 12,000 passes. Fig. 3 shows that each aggregate blend was able to meet

Fig. 2. J_c Values from SCB test.

Fig. 3. Loaded wheel tracking test results, 50°C, wet.

the dense mixture specification in the DOTD specification. The use of PG 67-22 binder resulted in three mixtures that were not able to meet the DOTD specification.

5.4. Balanced performance comparison

Fig. 4 presents the results from SCB and LWT together in the same graph. These comparisons included Louisiana state specification restrictions for balanced mixture design and thin lift mixture criteria. The goal is to have the mixture results located in the top left portion of the graph. Such samples will be rut and crack resistant. Four aggregate blends met the proposed requirements for

Table 3
Mixture design.

	Limestone	Gravel	Sandstone	Gravel+limestone	910LS	9.5 mm gravel	12.5 mm gravel
Gradations (percent passing)							
12.5 mm (1/2 in.)	100	100	100	100	100	100	97
9.5 mm (3/8 in.)	97	100	100	100	100	95	86
4.75 mm (No. 4)	92	97	93	100	100	85	66
2.36 mm (No. 8)	73	59	62	67	70	56	47
1.18 mm (No. 16)	53	38	40	40	46	41	35
0.6 mm (No.)	38	26	28	24	33	31	27
0.3 mm (No. 50)	25	15	22	15	21	20	16
0.15 mm (No. 100)	14	9	15	10	14	12	9
0.075 mm (No. 200)	9	6	9	7	11	8	6
Design criteria							
Asphalt content (%)	7.0	8.2	7.2	7.8	7.1	7	6.5
Air voids (%)	3.2	3.5	3.4	4.2	3.8	3.5	4
VMA (%)	17	15	14	18	17	14	14
VFA (%)	82	78	76	77	82	50	44
DP (Percent Dust)	1.6	1.7	2.0	1.5	1.6	2.6	2.1

Fig. 4. Balanced performance for mixtures.

a balanced mixture. The mixtures were comprised of PG76-22m asphalt binder, except for the sandstone aggregate blend. Three aggregate blends were rut prone but crack-resistant, LS- 67, 67-Gr+Ls, and 67-910. The 67-Gr mixture is rut resistant but prone to cracking. These results indicate that using a polymer modified binder can enhance the cracking or rutting issues on the 4.75 mm NMAS mixtures.

5.5. Dynamic modulus (E^*) test

Dynamic modulus was performed in accordance with AASHTO T342 to evaluate the general behavior of the 4.75 mm NMAS mixtures for pavement design purposes. This test determined the stiffness of the asphalt mixtures while treated by cyclic compressive loads. Master curves of performance have been established and shown in Fig. 5. Fig. 5 shows that mixtures are performing similarly with minor differences at low temperatures. However, the various aggregate structure results in variable high-temperature properties. While generally, the mixture with higher PG grade should perform stiffer, it was shown in Fig. 4 that aggregate type and aggregate design have a major effect on the elastic performances of mixtures with low aggregate sizes. The information developed by this testing will be used by the DOTD pavement design group to assign structural components, if any, to the new mixture design.

5.6. DOTD specification recommendation

Table 4 presents the recommended design specification developed for DOTD.

5.7. Cost and durability analysis

The specification was developed to assure the quality of mixtures. However, economic analysis is needed for comprehensive evaluation. Therefore, a cost analysis was conducted to present the economic differences between the mixtures with various NMAS and aggregate designs. Table 5 shows the costs per ton for each mixture while considering only one binder (PG 67-22) for all types. For cost analysis, the average price to produce the stockpile throughout the state was considered. This assumption and the tendency of lower-sized mixtures to require higher binder content to satisfy the volumetric and performance constraints caused the cost per ton of 4.75 mm NMAS mixtures to be slightly higher than the relative 9.5 and 12.5mm mixtures.

Table 4

DOTD 4.75 mm specification recommendation.

Gradations (percent passing)	
12.5 mm (1/2 in.)	100
9.5 mm (3/8 in.)	96-100
4.75 mm (No. 4)	90-96
2.36 mm (No. 8)	
1.18 mm (No. 16)	40-55
0.6 mm (No.)	
0.3 mm (No. 50)	
0.15 mm (No. 100)	
0.075 mm (No. 200)	6-12
Design criteria	
Air voids (%)	2.5-4.5
VMA (%)	>16
VFA (%)	>74
DP	1.0-2.0
Performance criteria	
LWT, rut depth, mm, 50°C, wet	12mm max @12,000 passes
SCB, J_c , 25°C kJ/m ²	0.5 min

Table 5

Production costs comparison per ton (materials only).

NMAS	Aggregate type				
	Gravel	Limestone	Gravel+LS	Sandstone	910s
4.75mm	\$44.29	\$58.10	\$54.63	\$48.72	\$61.24
9.5 mm	\$39.26	\$57.19	-	-	-
12.5 mm	\$37.18	\$55.69	-	-	-

The price per ton of 4.75 mm NMAS mixture showed to be higher than other mixtures. However, these mixtures will typically be placed on the roadway as a thin lift (< 19.0 mm). The analysis in Table 6 indicates a significant reduction in cost for 1 lane mile (3.66 m wide) construction with smaller aggregate mixtures. The yield for all the mixtures is constant, 23 N-m²-mm, due to similar design density.

5.8. Pavement performance analysis results

Fig. 6 presents pavement performance analysis using AASHTOWare Pavement-ME with respect to top-down cracking distress. Note that top-down cracking was considered the evaluating distress since it was the most dominant distress among all the distress types. Fig. 6 shows the pavement life, considering top-down cracking. A total of 10 asphalt pavements with various NMAS, including 4.75 mm, 9.5 mm, and 12.5 mm as well as different aggregate types, gravel, limestone, sandstone, 910

Fig. 5. Elastic modulus tests result for the mixtures.

Table 6
Comparison of mixture costs per lane mile.

NMAS	Thickness	Gravel	Limestone	Gravel + LS	Sandstone	910s
4.75mm	3/4"	\$12,862	\$16,872	\$15,865	\$14,148	\$17,784
4.75mm	1"	\$17,149	\$22,496	\$21,153	\$18,864	\$23,712
9.5 mm	1.5"	\$22,802	\$33,216	-	-	-
12.5 mm	2"	\$28,792	\$43,126	-	-	-

Table 7
Comparison of mixture costs per lane mile per operation year.

4.75 mm NMAS				9.5 mm NMAS		12.5 mm NMAS
Gravel	Limestone	Gravel + LS	Sandstone	910s	Gravel	Gravel
\$695	\$844	\$1024	\$707	\$1078	\$1471	\$1986

Fig. 6. Pavement life results based on top-down cracking.

limestone, and Gr+LS were considered for the analysis. It is noted that two different binder sources (B1 and B2) were used in the mixtures. Also, asphalt pavements with similar aggregate types were considered for the comparison.

In general, mixtures with higher dynamic modulus values showed lower lives. Comparing 67-LS and 76-LS pavements, mixtures containing binder PG 76-22 yielded lower fatigue life, indicating the adverse effect of stiffer material on pavement performance. Similarly, the gravel and Gr+LS mixtures with lower dynamic modulus values illustrated higher pavement lives as predicted by top-down cracking. Further, 9.5 mm and 12.5 mm pavements showed comparable fatigue lives compared to 4.75 mm mixtures. Although these pavements had higher thicknesses, a similar performance with Gr+LS pavements was observed.

Table 7 shows the cost of each mixture divided by the years of operation from Fig. 6. This information provides a clear comparison between different mixtures' cost and their representing performance life. It is noticeable that thicker mixtures cost more than 4.75 mm NMAS mixtures due to similar durability performance. Between the 4.75 mm NMAS mixtures Gravel and Sandstone mixtures are the most cost-effective mixtures

6. Conclusions

Based on the laboratory evaluation of the test results on different 4.75 NMAS mixtures and mechanistic-empirical design of pavements, the following conclusions were made about the effect of aggregate and binder types on performances of these mixtures.

1. 4.75mm NMAS mixtures generally exhibited acceptable SCB and LWT performance. All mixtures except 67-LS, 67-Gravel+LS, and 67-910s are passing the LWT requirement for the rutting. These results indicate that using a polymer

modified binder can enhance the cracking or rutting issues on the 4.75 mm NMAS mixtures.

2. While higher PG grade mixtures should perform stiffer in dynamic modulus test, it was observed that aggregate type and aggregate design have a major effect on the elastic performances of mixtures with low aggregate sizes. For pavement design practices, the department will need to develop a range of acceptable stiffness values.
3. Mechanistic-empirical analysis results showed that asphalt mixtures with higher dynamic modulus values yielded pavements with lower fatigue lives.
4. 4.75 NMAS pavements (Gr+LS) showed comparable fatigue lives as compared to 9.5 and 12.5 NMAS pavements.
5. The cost analysis of the mixtures showed considerable advantage of using 4.75 mm NMAS mixtures due to lower thickness application. This is fascinating outcome since the laboratory testing and Mechanistic-Empirical simulation showed comparable performance between lower aggregate size mixtures and the conventional mixtures.
6. Comparison between years of performance and cost per mile showed gravel and sandstone mixtures to be the most cost-effective mixtures.

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Conflicts of interest

The authors declare that there are no conflicts of interest regarding the publication of this paper.

Authors' contributions

The authors confirm contribution to the paper as follows: study conception and design: Saman Salari, Samuel Cooper, Louay Mohammad, Peyman Barghabany; data collection: Saman Salari, Samuel Cooper, and Peyman Barghabany; analysis and interpretation of results: Saman Salari, Samuel Cooper, and

Peyman Barghabany; draft manuscript preparation: Saman Salari, Samuel Cooper, Peyman Barghabany, Louay Mohammad. All authors reviewed the results and approved the final version of the manuscript.

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