NEW MEXICO DEPARTMENT OF TRANSPORTATION

RESEARCH BUREAU

Innovation in Transportation

Development of a Balanced Asphalt Mixture Design Procedure for New Mexico

Final Report

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Research Bureau
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Albuquerque, NM 87109

In Cooperation with:
The US Department of Transportation
Federal Highway Administration

Report NM18MSC-02

APRIL 2019

USDOT FHWA SUMMARY PAGE

1. Report No. NM18MSC-02	2. Recipient's Catalog No.
3. Title and Subtitle Development of a Balanced Asphalt Mixture Design Procedure for New Mexico	4. Report Date April 2019
5. Author(s): Rafiqul A. Tarefder and Mohiuddin Ahmad	6. Performing Organization Report No. NM18MSC-02
7. Performing Organization Name and Address University of New Mexico Department of Civil Construction and Environmental Engineering MSC01 1070 1 University of New Mexico Albuquerque, NM 87131	8. Performing Organization Code 456A
	9. Contract/Grant No. 456-567
10. Sponsoring Agency Name and Address Research Bureau New Mexico Department of Transportation (NMDOT) 7500B Pan American Freeway PO Box 94690 Albuquerque, NM 87199-4690	11. Type of Report and Period Covered Final Report April 10, 2018 – April 10, 2019
	12. Sponsoring Agency Code New Mexico DOT
13. Supplementary Notes	

14. Abstract

A balanced mix is a mix that satisfies both rutting and cracking criteria or limiting values. Though rutting criteria of NMDOT mixes was obtained from a recently completed project, the cracking criteria of NMDOT mixes were not available. Therefore, extensive literature was gathered in this study to come up with limiting values of different types of cracks. It was revealed that only a very few states have conducted cracking tests so far. Therefore, the cracking criteria employed in this study were not the final ones but gave the researchers a starting point for designing a balanced mix. The goal of this project was to collect aggregates and binders and design trial asphalt mixes and examine if one of them satisfied both rutting and cracking criteria. In this study, aggregates from one source were collected, and trial mixes were designed using one Performance Grade (PG) binder in the laboratory using the Superpave procedure. The mix was evaluated for balanced mix criteria. Rut testing was performed using the Hamburg Wheel Tracking Device (HWTD) and cracking tests were performed in Semicircular Bending (SCB) and Disk-Shaped Compact Tension (DCT) modes. To this date, it was observed that the mix does not satisfy some of the cracking criteria. A balanced mix could not yet have resulted from this study. It can be also noted that very recently an Overlay Tester (OT) was acquired through this project. Only a very few OT tests were conducted so far to gain experience and have confidence in the new test data.

15. Key Words Balanced Mix Design, Cracking, Rutting		16. Distribution Statement Available from NMDOT Research Bureau	
17. Security Classi. of the Report None	18. Security Classi. of this page None	19. Number of Pages 64	20. Price N/A

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PROJECT NO. NM18MSC-02

DEVELOPMENT OF A BALANCED ASPHALT MIXTURE DESIGN PROCEDURE FOR NEW MEXICO

FINAL REPORT

April 10, 2018 – April 10, 2019

A Report on Research Sponsored by:

Research Bureau
New Mexico Department of Transportation
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PREFACE

The research reported herein provides balanced mix design procedure and limiting values of different types of cracking. Some laboratory rutting and crack testing procedure, and data analyses are reported herein.

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ABSTRACT

A balanced mix is a mix that satisfies both rutting and cracking criteria or limiting values. Though rutting criteria of NMDOT mixes was obtained from a recently completed project, the cracking criteria of NMDOT mixes were not available. Therefore, extensive literature was gathered in this study to come up with limiting values of different types of cracks. It was revealed that only a very few states have conducted cracking tests so far. Therefore, the cracking criteria employed in this study were not the final ones but gave the researchers a starting point for designing a balanced mix. The goal of this project was to collect aggregates and binders and design trial asphalt mixes and examine if one of them satisfied both rutting and cracking criteria. In this study, aggregates from one source were collected, and trial mixes were designed using one Performance Grade (PG) binder in the laboratory using the Superpave procedure. The mix was evaluated for balanced mix criteria. Rut testing was performed using the Hamburg Wheel Tracking Device (HWTD) and cracking tests were performed in Semicircular Bending (SCB) and Disk-Shaped Compact Tension (DCT) modes. To this date, it was observed that the mix does not satisfy some of the cracking criteria. A balanced mix could not yet have resulted from this study. It can be also noted that very recently an Overlay Tester (OT) was acquired through this project. Only a very few OT tests were conducted so far to gain experience and have confidence in the new test data.

ACKNOWLEDGMENTS

This project was funded by the New Mexico Department of Transportation's (NMDOT) Research Bureau.

The authors would like to express their sincere gratitude and appreciation to James Gallegos, NMDOT Materials Bureau Chief, for being the sponsor of this project. The authors express their gratitude to Jeremy Rocha, Parveez Anwar (ret.) and Kelly Montoya for advocating the project. Special thanks go to Shawn Hammer of Fisher Sand & Gravel New Mexico, Inc. for supplying hundreds of bags of aggregate, mix and binders. The UNM research team appreciates the valuable service and time of the Project Manager, Mr. Virgil Valdez. The UNM research team would like to thank all the Project Technical panel members for their valuable suggestions during the quarterly meetings.

The authors would like to thank several members and personnel at UNM for their support.

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INTRODUCTION

RESEARCH NEED AND SIGNIFICANCE

The New Mexico Department of Transportation (NMDOT) has been using the Superpave mix design procedure for the past twenty years. There are several issues with the Superpave mix design procedure. It relies mostly on volumetrics, which are believed to take care of the field performances. However, there is no actual validation of the predicted performance with the field performances. Recently, Superpave pavements have shown to fail by premature cracking rather than rutting. Over the past years, a number of pavements were rehabilitated using mill and fill and overlays. These pavements have shown lot of reflective cracking. A study conducted by Shane Buchanan (*1*) showed that out of 40 pavements from 30 states, only 7% showed rutting whereas 53% suffered from longitudinal cracking and 44% suffered from reflective cracking. Because cracking provides pathways for water and air, it influences durability including raveling and potholes.

To eliminate the cracking issues associated with Superpave pavements, this study attempts to develop a mix design procedure that balances between rutting and cracking. The objective is to develop a well-performing mixture rather than an economic one as considered for Superpave mix design. This procedure is termed as Balanced Mix Design (BMD) approach. The long-term goal is to define a BMD mix design using performance tests on appropriately conditioned samples that address multiple modes of cracking taking into consideration of mix aging, Reclaimed Asphalt Pavements (RAP), traffic, climate and layer within the pavement structure.

GOALS AND OBJECTIVES

The primary objective of this study is to develop a balanced mix design procedure for New Mexico which will reduce cracks while keeping rutting within certain limits. The specific objectives are:

a) Evaluate the performance tests necessary to determine BMD performance parameters. Determine the limiting values of various cracking for use in BMD.
b) Determine optimum binder content and best gradation based on performance tests on the trial mixes.
c) Develop BMD procedure including volumetric parameters for different traffic volumes.
d) Document the literature, laboratory testing, and recommendations on BMD and provide an implementation plan for NMDOT in the form of reports and presentations.

REPORT ORGANIZATION

This report is comprised of four sections, as follows:

Section 1: Balanced Mix Design Procedure This section describes the different mix design procedures to perform a balanced mix design.

Section 2: Different Performance Tests This section summarizes different crack testing procedures and limiting cracking and rutting values for BMD.

Section 3: Developing BMD Procedure for New Mexico This section describes the tests performed in the laboratory to design a balanced mix starting from material collection to performance testing.
Section 4: Conclusions This section summarizes the finding of this study.

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BALANCED MIX DESIGN PROCEDURE

OLD MIX DESIGN PROCEDURE

The history of asphalt concrete mix design is a little over a century. Several methods have been developed over time. One method had some limitations which had been tried to overcome by the next method. However, none of the methods developed a mix design procedure that was economical and at the same time well-performing. In 1890, F.V. Green of the Barber Asphalt Paving Company published a specification for the construction of asphalt pavement although it doesn't include any mix design procedure (*2*). It used 12-15% asphalt, 70-83% Sand and 5 to 15% lime. In 1905, Richardson described two types of asphalt mixes: surfacing mixes and asphalt concrete. Surfacing mixture is a sand mix with 9 to 14% asphalt by weight. He described that asphalt should be sufficiently hot to be fluid. Asphalt concrete is more like the HMA. Richardson used the abbreviation VMA to describe voids in the mineral aggregate. He proposed the adjustment of VMA to include the correct amount of asphalt. He also proposed a fine graded mixture with a nominal maximum size of 1.5 in, an optimum bitumen content 7.4% and VMA of 13.2%. He noted that if in place air void is higher (say, 5-8%), the pavement would crack, although no field performance was described. Until the beginning of the 1920s, asphalt concrete was used as a base, and surfacing mixture was used as the surface.

Hubbard Field mix design procedure was developed in the 1920s by Asphalt Association and used until 1960s. They developed a laboratory compaction procedure for the surface mix. A modified Hubbard Field method also described the laboratory sample preparation procedure for asphalt concrete. This method first described the evaluation of air voids and the stability of the mix. Depending on air void and stability, this method described a procedure to select a design asphalt content.

From 1927 to 1940s, Hveem conducted several studies to determine the surface area of aggregate, the stability of the mix, and a mix design procedure based on this. Aggregate properties and binder content together influence the stability and cohesion. For durability, Hveem developed a method to predict moisture susceptibility. Hveem didn't consider air void as a mix design parameter. A study of Hveem pavements reveled that they were dry mix, and fatigue crack was predominant.

Bruce Marshal developed a mix design method in the late 1930s. It is similar to the Hveem method with differences in sample preparation technique. Marshal incorporated air void in mix design although didn't consider VMA. He considered Voids Filled with Asphalt (VFA) as a major design parameter. Later McLeod modified Marshal method by incorporating VMA. Later on, L.C. Krchma stated that film thickness as in Hveem method is an important factor for mix design. It was observed that VMA varies depending on nominal maximum aggregate size, no correlation between VMA and surface area was established.

SUPERPAVE MIX DESIGN PROCEDURE

Old mix design procedures only considered volumetric with little to no inclusion of performance. Also, old design methods didn't include a compaction procedure that is representative of the field. Those procedures didn't consider climate and traffic as a factor. Old methods did not have any procedure to predict the performance of the pavement using that specific mix design procedure.

To overcome these issues, Strategic Highway Research Program (SHRP) took the initiative to improve material selection and mixture design by developing a new mix design method that also uses asphalt binder evaluation. Finally, in 1993, they developed a superior performing asphalt pavement system (Superpave). Although Superpave mix design procedure brought huge promise to the asphalt pavement industry, recent studies found premature damage of the pavements constructed with Superpave method. Most of the pavements showed longitudinal or reflection cracking, and few have shown rutting. Therefore, agencies are trying to find an alternate option or improvement of the Superpave method so that cracking of the pavement is minimized while keeping rut within a tolerable limit. This may increase initial construction cost; however, it will reduce reconstruction or maintenance cost significantly in the long run.

BALANCED MIX DESIGN PROCEDURE

As Superpave mix design procedure failed to address the pavement distresses especially longitudinal, and reflection cracking, researcher around the country have recently started to modify their mix design procedures by adding some laboratory evaluation tests so that the field distresses can be addressed.

There two sides of BMD: stability and durability. Stability includes rutting, shoving, and flushing whereas durability includes cracks, raveling, and permeability. Most studies found that effective binder V_{be} is the primary mixture design parameter that affects both fatigue cracking. Different agencies are trying to decrease N_{des} to increase V_{be}. However, it does not work all the time. Stability is mostly evaluated using a Hamburg wheel tracking device. Based on material type and traffic, a specification value for rutting can be obtained. However, not all states use HWTD for rut or stability evaluation. About 40% of them use only volumetric, 35% do Hamburg wheel, 33% uses APA, 5% uses AMPT flow number, and 5% uses AMPT dynamic modulus to evaluate the stability of a mix.

Durability mainly measures the modes of cracking considering the aging condition of the pavement. Although there are some test methods available to evaluate cracking potentials, none of them is a very good predictor of field cracking. Three cracks are mainly considered in BMD durability evaluation: fatigue cracking, low-temperature cracking, and reflection cracking. Equipment used are Disk-shaped Compact Tension (DCT), Semi-Circular Bending (SCB), Texas Overlay Test (OT), Indirect tensile teste (IDT) and four-point bending. The NCHRP 9-57 contains a list of equipment should be used to evaluate durability. It is observed from the synthesis that 72% of the agencies use volumetric, 12% use IDT, 7% use beam fatigue, 7% use Texas overlay, 7% use IDT fracture strength, 2% use DCT, 2% use Thermal Strain Restrained Stress Test (TSRST), 2% use direct tension (DT).

Illinois uses volumetric analysis for low temperature cracking, SCB for intermediate temperature cracking and Hamburg Wheel Tracking Test (HWTT) for high-temperature rutting. Wisconsin provide a better insight of BMD depending on the type of road and based on the purpose of the mix. It uses DCT for low temperature cracking (-18 to -24 ℃), SCB for fatigue at intermediate temperature, and HWTT at 50 ℃ for rutting. Wisconsin defines four types of mix and their types of usage. Mix with high rut and low cycles to fatigue cracking is defined as poor mix and it is suitable for non-surface temporary usage. Mix with low rut and low fatigue resistance is defined as stiff mix, which is suitable for the bottom layer of a full depth pavement. It is actually a dry mix. Mix with high rutting and high fatigue resistance is termed as soft mix suitable for reflective crack control. The mix with low rutting and high fatigue resistance is used for high traffic roads and as a surface mix. However, it is necessary to define how much rut is considered as high rut and how much fatigue resistance is considered as high fatigue resistance, and how to control them during mix design.

Balanced Mix Design Task Force was formed in 2015. They provided different BMD procedures. Different agencies adopted the suitable one based on their materials, climate condition and purpose of the traffic. There are three different ways to develop a BMD (*1*):

Approach 1: Volumetric Design with Performance Verification

Perform the volumetric mix design and determine an optimum binder content, followed by performance testing for rutting and cracking. If the criteria are not met, redesign the mix until the cracking and rutting criteria are met. Illinois, Louisiana, New Jersey, Texas, and Wisconsin started evaluating this procedure.

Approach 2: Performance Modified Volumetric Design

Perform the volumetric mix design and determine an optimum binder content followed by performance testing for rutting and cracking. If the criteria are not met, binder content is increased by 0.5%, until the cracking and rutting criteria are satisfied. Then analysis must be performed to make sure the volumetric are within the limit. California follows this procedure.

Approach 3: Performance Design

Performance tests will be performed at the beginning, and volumetric performance tests will be performed at the end. It is not necessary to satisfy the volumetric criteria. New Jersey has proposed this procedure.

Figure 1 shows the different BMD procedure, and Figure 2 shows the different states which have adopted these procedures.

Approach 1 includes redesigning the mix several times using the Superpave procedure until the performance criteria are met, which is very time-consuming. Approach 2 is very straight forward and may require less time than approach 1. Approach 3 is very unorganized. Considering time constraints and to develop a step by step procedure for BMD, approach two was adopted for this study in this phase.

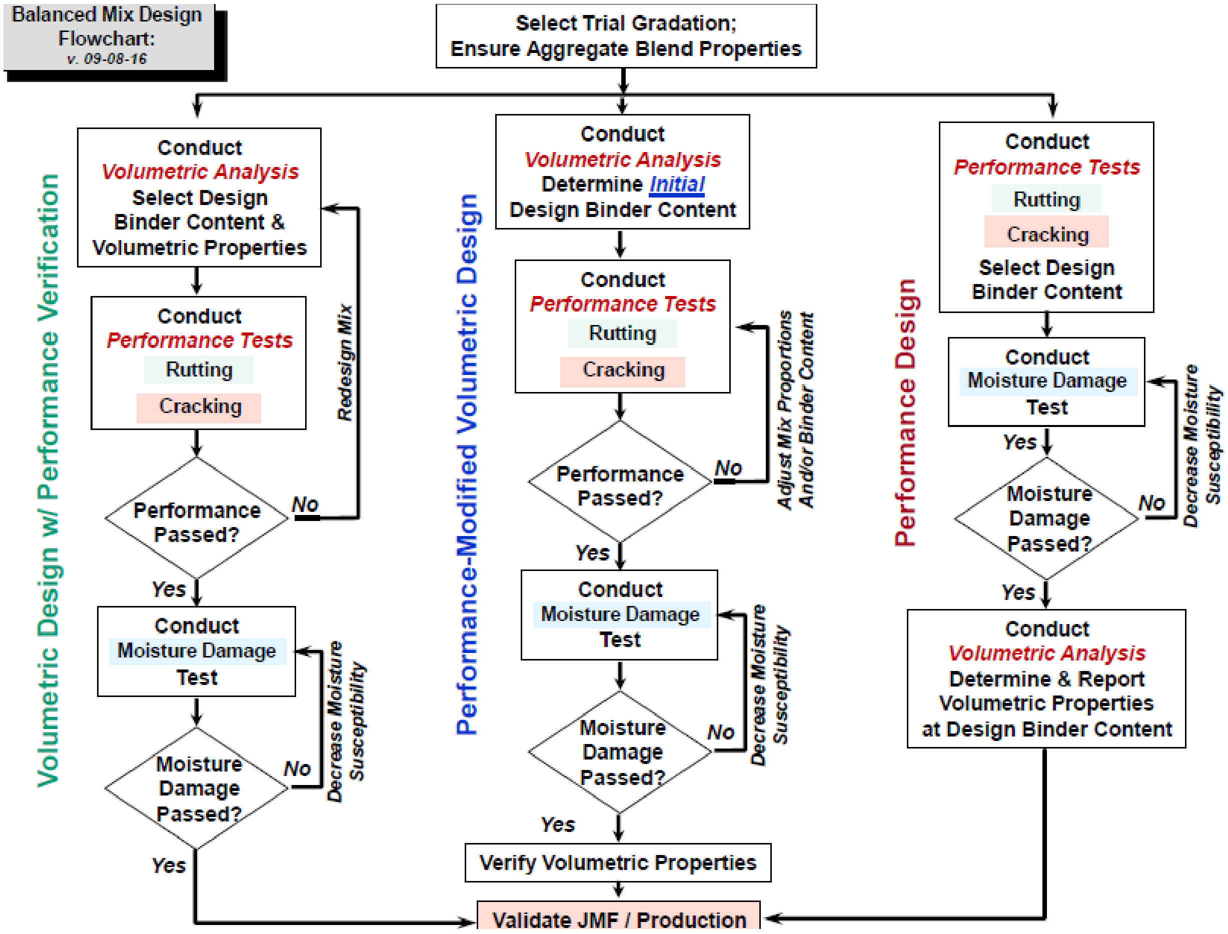

FIGURE 1 Samples of polymers used in asphalt modification

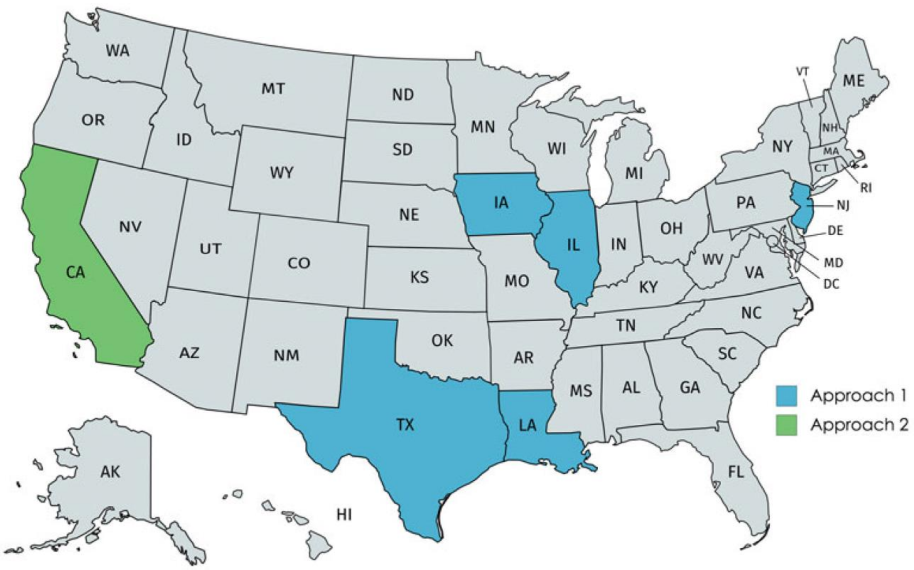

FIGURE 2 Different BMD procedures adopted by different states (Courtesy: NCAT)

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DIFFERENT PERFORMANCE TESTS

INTRODUCTION

Several states started conducting performance tests during a mix design to ensure mix performances. Although some knowledge on BMD has been developed so far, BMD lacks in the following areas:

1. For each mode of pavement cracking, what are the best cracking tests?
2. What mixture conditioning or aging protocols should be used for a specific performance test? Note that aging phenomena is complex and integral to cracking.
3. What are the or limiting values of cracking that should be used to design a BMD?

Different cracking tests were developed by many researchers. They are tests for thermal cracking, reflection cracking, bottom-up cracking, top-down cracking. To include those in the BMD procedure, the correlation between laboratory performances and field performances need to be evaluated. In addition, the following parameters need to be considered: testing time, analysis complexity, test variability, equipment variability and cost, and sensitivity to mixing design parameters.

As aging plays an important role in cracking, it is necessary to select an appropriate mixture aging condition or protocol for a specific cracking test. Rutting and moisture damage tests are usually performed on short term aged specimens, because AC pavements are mainly vulnerable to those distresses immediately after construction. On the other hand, mixtures tend to be more susceptible to cracking after aging due to reduced flexibility or brittleness. Therefore, long-term aged samples are preferred. In this study, we used the AASHTO R30 for aging a sample before cracking test (*3*).

BMD is a new procedure, and research is ongoing by different States and few leading states are listed in Table 1 below (*4*-*10*):

TABLE 1 Ongoing projects by different DOT's on BMD

Agency	Project
Caltrans	Simplified performance-based specifications for AC Long Life Projects
Idaho Trans Dept	Development and evaluation of performance measures to augment asphalt mix design in Idaho
Purdue Uni/IDOT	Performance Balanced mix design for Indiana's asphalt pavements
MnDOT	Balanced design of asphalt mixtures
TxDOT	Develop guidelines and design program for HMA containing RAP, RAS, and other additives through a BMD process

Based on literature, the following tests were identified in this study as a good candidate for evaluating BMD.

- HWTT test for rutting.
- SCB test for top-down cracking at intermediate temperatures.

- DCT or TSRST test for low-temperature cracking.
- Beam fatigue test for medium temperature bottom-up fatigue cracking.
- Texas Overlay test for reflective cracking.

A description of these test procedures and limiting values suggested by different states are described below.

HAMBURG WHEEL TRACKING TEST (HWTT)

Recently NMDOT developed a standard specification to test for rutting using HWTT test. The test apparatus is shown in Figure 3, and the sample dimension is shown in Figure 4. The air void of the specimen is 6 ± 1 %. The test is conducted at 50 ± 1 ºC. A 158 lb steel wheel passes over the sample at 52 passes per minute. The rut depth is measured for 20,000 cycles.

a) Wheel tracking test (before) b) Wheel tracking test (after)

FIGURE 3 HWTT test

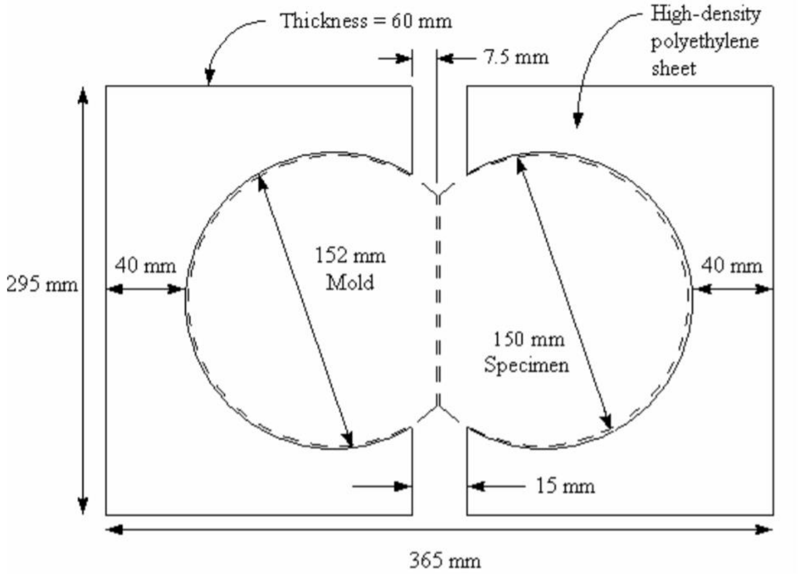


FIGURE 4 Dimension of the test specimen

NMDOT specification limits for different types of binders are shown in Table 2. To design any mix in New Mexico, these criteria must be satisfied.

Table 2 HWTT Specifications for New Mexico

Binder Grade	Number of Wheel Passes	Test Temperature (°C)	Max Rut Value (SP-III) (mm)	Max Rut Value (SP-IV) (mm)
64-XX	10000	50	5.5	5.5
	15000		7.0	6.0
	20000		11.5	6.5
70-XX	10000	50	4.5	4.5
	15000		5.5	5.0
	20000		7.0	5.5
76-XX	10000	50	3.5	3.5
	15000		4.0	4.0
	20000		5.0	4.5

SEMI-CIRCULAR BENDING (SCB) TEST

SCB test can be performed both in a monotonic or cyclic mode. Table 3 shows the spec followed by TxDOT (*11*). The ASTM recommended loading rate is 0.5mm/min. The distance between the supports is 0.8 times the specimen diameter. Walubita et al. (*12*) showed Coefficient of Variance (COV) for SCB is less than 20%. But TxDOT allows up to 30% COV for SCB test. Unlike the OT test, there are not enough studies to evaluate the factors that affect the COV of SCB test data.

TABLE 3 Test Configuration for Monotonic and Cyclic SCB Test

	Monotonic SCB	Cyclic SCB
Sample Dimension	6″ dia x 3″ H x 2.5″ T	6″ dia x 3″ H x 2.5″ T
Test parameters	Loading rate 0.05″/min, 77 ºF	f = 1 Hz, 77 ºF, Load = 50% of the peak load
Output data	σ_t, ε_t, E_t, G_f, *and FE index*	Number of cycles to failure
Test time	≤10 minutes	≤180 minutes

SBC data can be analyzed to determine Fracture Energy (FE) Index.

$$\sigma_t = 4.263\frac{P_{max}}{tD} \tag{1}$$

and

The tensile strain at the peak load or the ductility potential is measured as:

$$\varepsilon_t = \frac{D_{Pmax}-D_0}{D_0} \tag{2}$$

The tensile modulus or the stiffness in tension

$$E_t = \frac{\sigma_t}{\varepsilon_t} \tag{3}$$

The fracture energy is defined as the work required to produce a crack of unit surface area, measured in J/m^2. The work required to fracture the sample is represented by the area under the load versus displacement curve. Therefore, a general expression for fracture energy can be written as:

$$G_f = \frac{1}{A}\int f(x)dx \tag{4}$$

FE index is defined as a parametric ratio of the fracture energy to the tensile strength and tensile strain at peak failure load per unit crack length:

$$FE\ index = 1 \times 10^3 \frac{G_f}{l_{cr}\sigma_t}\varepsilon_t \tag{5}$$

where l_{cr} is the length traversed by the crack (the specimen thickness t, in case of the monotonic test).

Huang et al. (*13*) used Paris' law to represent cyclic SCB test results. Both crack initiation and propagation can be represented by the SCB test result. Fatigue crack damage in asphalt paving mixtures consists of two processes – crack initiation and crack propagation. Common approaches to characterize fatigue damage in asphalt mixtures include a phenomenological method and fracture mechanics' approach. In the phenomenological aprpoach, stress or strain cycles necessary to induce fatigue failure are applied to initially smooth-surfaced specimens under stress or strain-controlled mode. The fatigue life consists of the number of load cycles to initiate a dominant crack and to propagate this dominant crack to failure. In the fracture mechanics approach, the basic premise is that all engineering materials are inherently flawed. The fatigue life is the number of load applications to propagate a dominant flaw, which is measured form an initial size to a critical dimension. The fracture mechanics approach uses an empirical crack growth model based on Paris' law.

Another parameter that is used to represent monotonic SCB result is *J-integral* which is also known as the critical fracture energy release rate (*14*). The higher the value, the more the mix's resistance to fracture. Strain energy is determined at three different notch depth and from the plot, the slope is determined. *J-integral* is determined using the following equation:

$$J_c = -\frac{1}{b}\left(\frac{dU}{da}\right) \tag{6}$$

where, J_c = *J-integral* or critical strain energy release rate (kJ/m^2); b = sample thickness (m); a = notch depth (m); U = strain energy to failure (kJ); and dU/da = change of strain energy with notch depth (kJ/m).

Research studies also found the *J-integral* to be a reproducible parameter using the monotonic SCB test. It was reported in literature that the J_c obtained from SCB tests exhibits good correlations ($R^2 = 0.58$) with the field fatigue cracking, which indicates that J_c is a promising index (*14*). Louisiana reported an average J_c value of 0.48 for 51 mixes. SCB can be performed on both 4 in.

and 6 in. diameter samples. Both *J-integral* and K_{1C} is used to characterize fracture resistance. However, K_{1C} is used for linear material. Therefore, *J-integral* is preferred for Asphalt Concrete (AC) as it considers elasto-visco-plastic mechanism. The specific value of J-integral for Louisiana is 0.5-0.6 kJ/m^2. Also, *J- integral* is found to produce consistent results than K_{1C} value (*15*). Mixture with higher tensile strength usually has lower fracture resistance due to brittleness. ASTM standard for *J-integral* is 0.6 kJ/m^2 (*16*). Table 4 shows the summery of test procedures and specification values by different agencies.

TABLE 4 Summery of SCB test procedure and specification of different agencies

Agency	Spec title	Sample size	Temp	Loading rate	Other config.	Outcome	Spec value	Max COV
ASTM (*11*)	D8044-16	150mm dia, 57mm thick	(HT+LT)/2+4 ºC	0.5mm/min	25, 32, and 38 mm notch 7±0.5% Va	J-integral	0.5-0.6 kJ/m^2	
AASHTO (*17*)	TP 105	150mm dia, 25±2 mm thickness	PG lower limit+10 ºC	0.03mm/min CMOD	15mm notch,	Fracture energy and toughness		20%
LA (*18*)	ASTM D8044-16	150mm dia, 57mm thick	(HT+LT)/2+4 ºC	0.5mm/min	25.4, 31.8, and 38.1 mm notch 7±0.5% Va	J-integral	0.6 for PG76 and above and > 3MESAL; 0.5 for below PG76 and < 3MESAL	
IL		150mm dia, 50 mm thick	77 °F	50mm/min	15mm notch length	Flexibility Index		
MN		150mm dia, 25 mm thick	PG lower limit+10 ºC	0.03mm/min CMOD	15 mm notch length	Fracture energy, toughness	FIVE values of 230 J/m^2 for CIR	
NE (*19*)		150mm dia, 40-60mm thick	21 °C	1-5 mm/min	4±0.5% Va Notch length 15mm	Fracture energy		15%
WisDOT (*20*)	AASHTO XXXX	150 mm dia, 57 mm thick	25 °C	0.5 mm/min	25.4, 31.8, and 38.1 mm notch 7±0.5% Va	*J-integral*		

DISK-SHAPED COMPACT TENSION (DCT) TEST

Wagoner et al. (*21*) developed the DCT test as a method for obtaining the fracture energy of asphalt concrete (HMA) following the ASTM E399 (*22*) Standard Test Method for obtaining plain-strain fracture toughness of metallic specimens.

The DCT as shown in Figure 5 is specified in the ASTM procedure D7313(*23*). The test is generally used to obtain the fracture energy, which can be used in performance-type specifications to control various forms of cracking, such as thermal, reflective, and block cracking of pavements surfaced with AC. Standard testing is conducted at 10 °C warmer than the PG low-temperature grade.

(a) A prepared sample for DCT test

(b) DCT test in progress

(c) Fracture face after DCT test

FIGURE 5 DCT test configuration

The DCT test is run in Crack Mouth Opening Displacement (CMOD) control mode at a rate of 1 mm/min. Typically, specimens are completely failed in the range of 1 to 6 mm of CMOD travel. Although the actual test takes only 1 to 6 minutes to perform, the actual amount of testing time per specimen is about 15 minutes, accounting for stabilization of test temperature, loading samples into the test apparatus, etc.

Fracture energy is calculated using the Eq. (7) below:

$$G_f = \frac{Area}{B(W-a)} \tag{7}$$

where, G_f = fracture energy (*kJ/m*^{2}); *Area* = area under the CMOD curve; *B* = specimen thickness; and *W-a* = initial ligament length.

Minnesota specified a limit for fracture energy is 0.4 *kJ/m*^{2} for low, 0.45 *kJ/m*^{2} for moderate, and 0.6 *kJ/m*^{2} for high traffic level (*24*). Their field cracking performance showed good correlation with fracture energy. Fracture energy increases with the increase in binder content up to an optimum point. Louisiana has a different specification for fracture energy as shown in Table 5. For WisDOT, the values are different as shown in Table 6 (*25-26*).

TABLE 5 Minimum design G_f values for LTC

Traffic level	G_f (*kJ/m*^{2})
Low (<10M ESALs)	0.4
Medium (10-30 M ESALs)	0.46
High (>30M ESALs)	0.69

TABLE 6 DCT energy (*kJ/m*^{2}) requirement for WisDOT

	Low traffic	Medium traffic	Heavy traffic
DCT short term aged	0.3	0.4	0.5
DCT long term aged	0.25	0.3	0.35

The variability of DCT is lower compared to other fracture tests. Mandal et al. (*26*) obtained a standard deviation of the test results is 75 *J/m*^{2} which is the same as ASTM standard of 78.5 *j/m*^{2}. Wagoner et al. (*27*) obtain COV of DCT test ranging from 3 to 28%. Table 7 summarizes the test parameters as well as an outcome from the DCT test from different agencies.

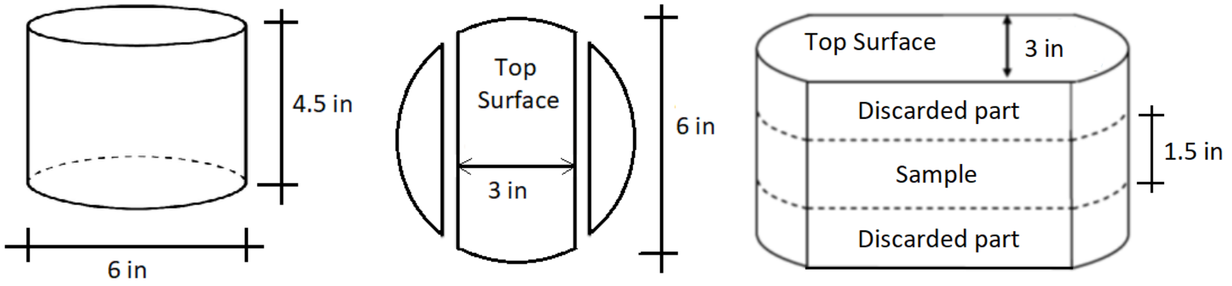
TABLE 7 DCT test parameters and outcomes

Agency	Spec title	Sample size	Temp	Loading rate	Other config.	Spec value	Max COV
ASTM	D7313-13	50±5mm thick, 150mm dia	10 °C above lower grade	1mm/min, CMOD opening	Notch 62.5±5 mm		78.5 J/m^2 of variation
LA	D7313-13	50±5mm thick, 150mm dia	10 °C above lower grade	1 mm/min, CMOD opening	Notch 62.5±5 mm	690 J/m-2 for >30MESAL, 460 for 4-30M, 400 for <3MESAL	15%
MN (*28*)		50±5mm thick, 150mm Dia	10 °C above lower grade	1 mm/min CMOD	62.5±5 mm notch and 7% Va	400 J/m^2 for L, 460 for M and 690 for high traffic	Allowable difference 90 J/m^2

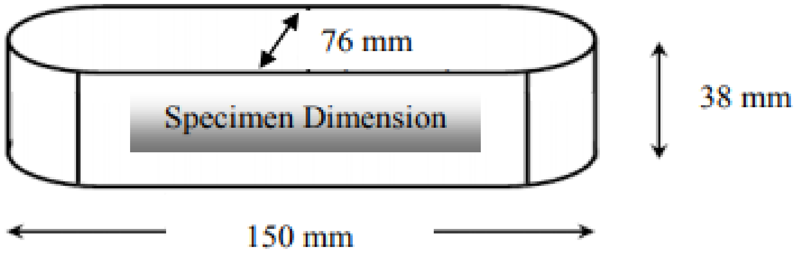
WisDOT (*29*)	Modified D7313-13	50±5mm thick, 150mm dia	10 °C above lower grade	1 mm/min, CMOD opening	Notch 62.5±5 mm 6.5±0.5% Va for 3.5% JMF and collected mix, 7±0.5% for 4% JMF,		78.5 j/m^2 variation
IlliTC	D7313-13	50±5mm thick, 150mm dia	10 °C above lower grade	1 mm/min, CMOD opening	Notch 62.5±5 mm	400 J/m^2	15%

TEXAS OVERLAY TEST (OT)

OT test is used to evaluate the reflective cracking potential of asphalt mix. The overlay tester can effectively differentiate between the reflection cracking resistance of different asphalt mixtures (*30*) and shows good correlations with field cracking. The test is conducted as per Tex-248-F procedure (*31*). Samples are compacted and trimmed as the dimensions shown in Figure 6.



a) Compacted sample b) Side trimming c) Height trimming



d) Final trimmed sample

FIGURE 6 Dimension of the Texas OT test

The opening between two metal plates is 2 mm, which works as a crack opening. The test temperature is 77±1 ºF. A cyclic displacement with the max magnitude of 0.025 inches is applied at 10 sec/cycle rate. A total of 0.025 inch is selected based on the displacement experienced by a concrete pavement undergoing 20 ºC change in temperature with a 4.5 m joint or crack spacing (*30*). The failure criteria are 93% reduction of the initial load. TxDOT specification value is 300 cycles (*32*).

Data from Texas OT tester varies significantly. Therefore, a COV value of 30% or less is considered as acceptable by TxDOT. Studies have been conducted by many researchers to identify factors that contribute to the changes in COV as well as the cracking resistance (*33*-*36*).

Effects of Sample Thickness

The effects of sample thickness on OT results is shown in Table 8. It is observed that the COV is minimum for a 1.5-inch-thick sample. Number of cycles to failure increases exponentially with an increase of thickness.

TABLE 8 Effect of thickness on OT cycles and COV

Thickness	OT cycles (3 out of 5)		
	With limestone	Limestone + RAP	Quartz + RAP
1			67
1.5	28	9	304
2	237	347	1000+
2.5	1000+	970	
Thickness	COV		
1			23%
1.5	22%	12%	11%
2	19%	18%	
2.5		16%	

Effects of Loading

The magnitude and frequency of cyclic deformation affect the OT cycles and COV values. Figure 7 and 8 represent the changes in COV and OT cycles with deformation and frequency respectively. It is observed that COV is minimum for a displacement of 0.025 in. and frequency of 10 Hz.

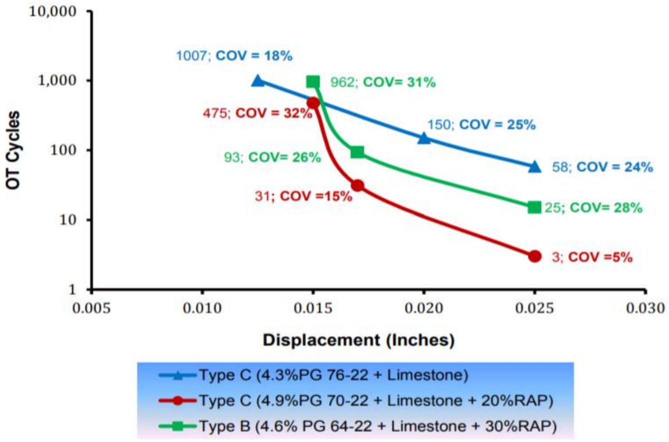


FIGURE 7 Changes in OT cycles and COV with displacement magnitude

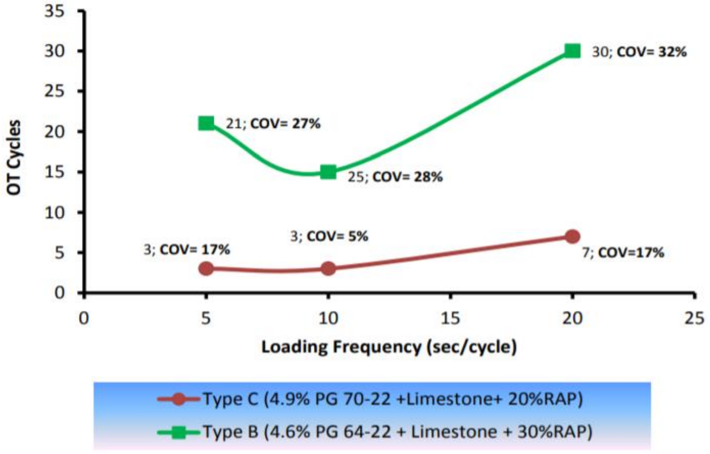


FIGURE 8 Changes of OT cycles and COV with loading frequency

Effects of Aggregate Gradation

As shown in Figure 9, coarse graded mix shows multiple cracks as crack tip tries to find a week zone to propagate the crack and changes its direction when crack tip hits a solid rock face. Therefore, the OT test results for coarse mixes are highly variable compared to the fine graded mix. Tex-248 protocol requires the recording of crack numbers. If five samples have COV of less than 30%, the samples with multiple cracks will be discarded.

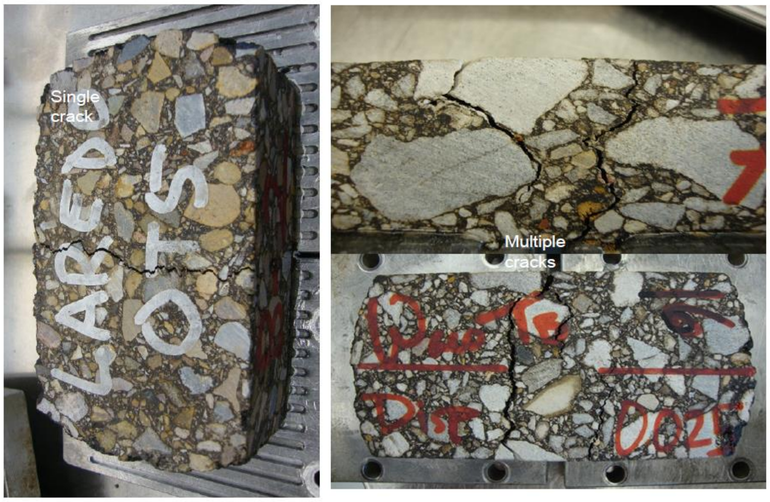


FIGURE 9 Single and multiple crack path

Overall, the COV of the OT test can be minimized if the following procedure is followed:

1. Table 9 should be followed.
2. The OT machines should be periodically checked and calibrated for both load and displacement measurements. This aspect is discussed in the subsequent sections of this chapter.
3. All technicians and operators running the OT machines must be trained and certified. For starters, at least two technicians/operators must be trained per lab and a certification

program initiated. This OT certification program can either be an online course along with video demos or following the format of the TxDOT standard certification training programs.

4. Updated software improved the test result by reducing the COV.
5. The drying process affects the test results significantly. The best drying method is, therefore, to use oven drying at 104F for a minimum period of 12 hours to constant weight but not to exceed 24 hours.
6. Variability in the results is within acceptable limits (i.e., COV<30 percent) if all the samples are tested within five days of molding. Variability becomes very high when some of the replicate samples are rested for up to 7 days or more before testing.
7. OT specimens having AV values between 6.5 percent and 7.5 percent are the most repeatable with the lowest COV values.
8. The glue type and quantity also affect the COV of test results. The table below compares different glues, and the amount should be used.
9. COV doesn't show any good trend with the increase in temperature.
10. Since changing the loading rate does not improve repeatability nor reduce variability in the OT test results and the fact that there is lack of field validation to support the proposal to the modified loading parameters; these researchers recommend maintaining the current load settings of 0.025 in. Opening displacement and 10 sec/cycle.
11. The OT result variability does not show any definitive trend of variation with changing opening displacement and using the currently practiced 0.025 in. Opening displacement is recommended.

TABLE 9 Procedure to follow to optimize COV

Factor	Variable	Recommendation
Operator effect	Trained or untrained	Trains at least two techs per lab
Sample replicates	3, 4, and 5	Test 5 and choose the best 3 for lowest COV
Drying method	Air, oven, or core dry	Oven dry at 104 ºF >12 hours to a constant weight
Sample molding size	Different compaction height (2.5 to 5 inch)	Mold 5-inch-tall and cut two specimens from the middle
Sample sitting time	3 to 20 days	Test within five days
Glue type	Different type of putty	Devcon 2-ton 2—part epoxy at 2500 psi strength < 8 hours curing time to full strength
Glue quantity	12-18 grams	Use 14±2 grans
Air void	5 to 9%	Use 7±1%, target 7±0.5% if possible
Test temperature	73 to 81 ºF	Use 77±2 ºF
Wait time after mounting the sample	5 to 30 min	≥ 10 minutes
Loading displacement	0.0125 to 0.025 inch	Continue using 0.025 inch
Loading frequency	5 to 20 sec/ cycle	Use 10 sec/ cycle
Sample thickness	1 to 2.5 inch thick	Stick with current Tex-248 -F spec
Notching	0 and 0.25-inch notch	No notching is recommended

Number of Samples Required to Optimize COV

TxDOT provided a procedure to evaluate the number of samples required for a desired level of accuracy. This is a classic application of confidence intervals in statistical analysis. For a known population variance, the number of replicates required to achieve the specified levels of tolerance and reliability is defined in the following well-known Eq. (8):

$$n = \left(\frac{Zs}{\Delta x}\right)^2 \tag{8}$$

where,

n = number of specimens; Z = two tailed probability static from the standard normal distribution; s = population standard deviation; and Δx = specific tolerance value.

Three tests for each mix is recommended, which will give error less than 10% as shown in Figure 10. However, most of the time, there is one outlier. Therefore, at least four samples should be tested, and five samples are a good number. Past studies showed that if three best results from five are considered, the COV value is within the tolerable limit of 30% (*37*).

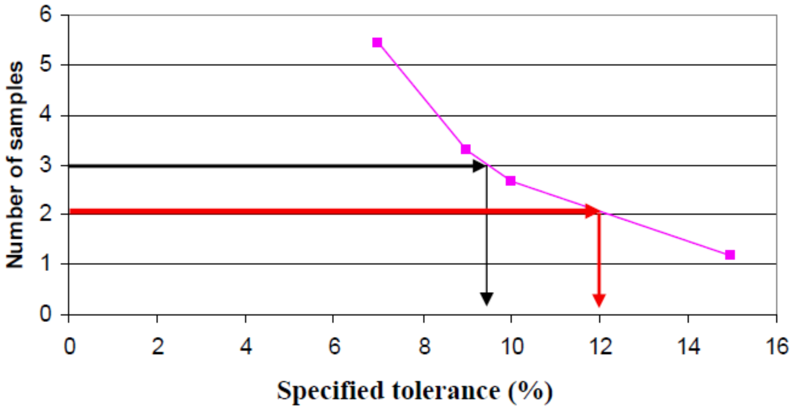


FIGURE 10 Changes in error with the number of samples

Monotonic OT

Like the cyclic OT test, monotonic OT test can be performed. However, fracture energy, stress or strain doesn't show any good correlation with OT cycles or field performance. TxDOT is using Fracture Energy (FE) Index, a mathematical parameter that includes fracture energy with stress and strain. This parameter shows a good correlation with OT cycles. The process to determine FE index using load-deformation plot (Figure 11) is described below.

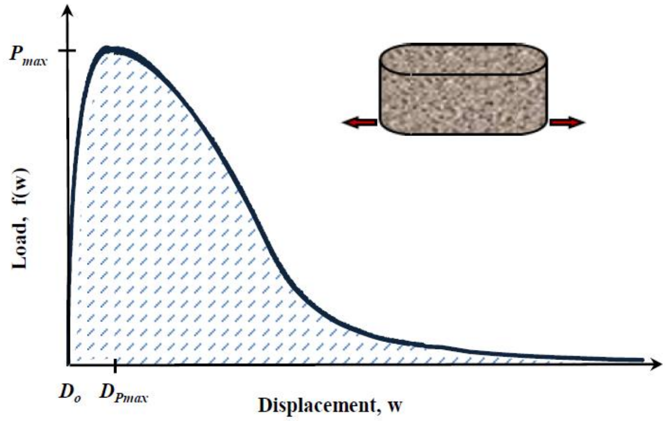


FIGURE 11 Load-deformation relation for monotonic OT test

The tensile strength is calculated as

$$\sigma_t = \frac{P_{max}}{tb} \tag{9}$$

where, *t* and *b* are the thickness and width of the sample.

The tensile strain at peak load or the ductility potential is measured as:

$$\varepsilon_t = \frac{D_{Pmax} - D_0}{D_0} \tag{10}$$

The tensile modulus or the stiffness in tension

$$E_t = \frac{\sigma_t}{\varepsilon_t} \tag{11}$$

The fracture energy is defined as the work required to produce a crack of unit surface area, measured in J/m^2. The work required to fracture the sample is represented by the area under the load versus displacement curve. Therefore, a general expression for Fracture Energy can be written as:

$$G_f = \frac{1}{A} \int f(x)dx \tag{12}$$

Finally, the FE index is defined as a parametric ratio of the fracture energy to the HMA tensile strength and tensile strain at peak failure load per unit crack length

$$FE\ index = 1 \times 10^3 \frac{G_f}{l_{cr}\sigma_t} \varepsilon_t \tag{13}$$

where *lcr* is the length traversed by the crack (the specimen thickness *t*, in case of the OTM test).

Table 10 describes the differences in test procedure between monotonic and cyclic OT test and Table 11 describe the benefit of monotonic OT over cyclic OT. FE Index values correlate to the perceived performance of their corresponding mixes and the OT cycles, whereas the fracture energy, tensile strength, and tensile strain parameters show no justifiable trend. Note that by comparison, the monotonic loading OT test is a much shorter test to run than the repeated loading OT test—an average of 5 to 10 minutes test time (monotonic loading OT) versus a test time range of 30 minutes to as much as over 3 hours (repeated loading OT), depending on the HMA mix type. Figure 12 shows the correlation between OT cycles to 93% with EI index.

TABLE 10 Comparison between monotonic and cyclic OT test procedure

	Monotonic OT	Cyclic OT
Sample dimensions	6″ L x 3″ W x 1.5″ T	6″ L x 3″ W x 1.5″ T
Test parameters	Loading rate 0.125″/min, 77 ºF	f = 0.1 Hz, loading rate 0.025″ max displacement, 77 ºF
Output data	σ_t, ε_t, E_t, G_f, *and FE index*	Number of cycles with peak load reduction
Test time per specimen	≤10 minutes	≤180 minutes

TABLE 11 Benefits of monotonic OT over cyclic OT test

Category	Monotonic OT	Cyclic OT
Sample preparation	Easy	Easy
Potential to test field cores	Yes	Yes
Test simplicity	Very simple	Very simple
Test time	≤10 minutes	≤180 minutes
Test variability	Repeatable	Repeatable
Correlation to field data	Need validation	Yes

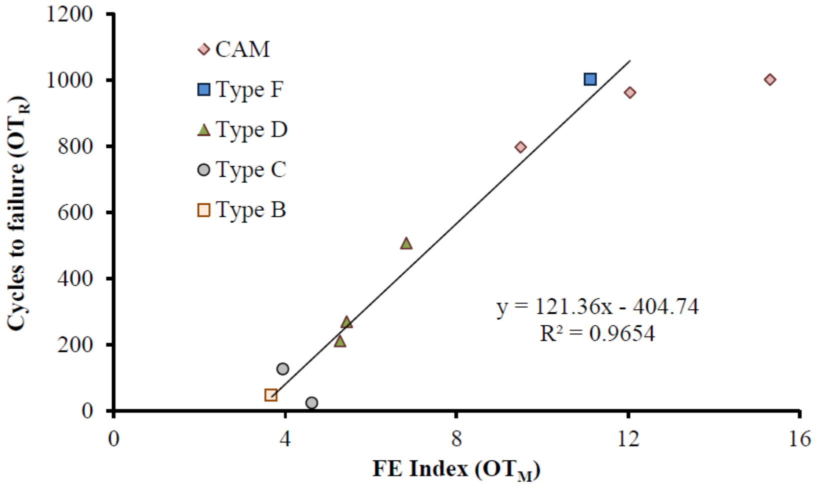


FIGURE 12 Correlations between OT cycles and FE index

Determining Fracture Parameter from OT Test

The flowchart to evaluate the fracture parameter of Paris' law (*A* and *n*) is shown in Figure 13. As it is difficult to measure crack length continuously with a number of cycles, load reduction with the number of cycles is evaluated. Load reduction is related to a number of cycles by regression analysis. Two parameters from this plot are evaluated as *A'* and *n'*. Later they are used to determine the fracture parameter *A* and *n*. Summery of OT test from different agencies are shown in Table 12.

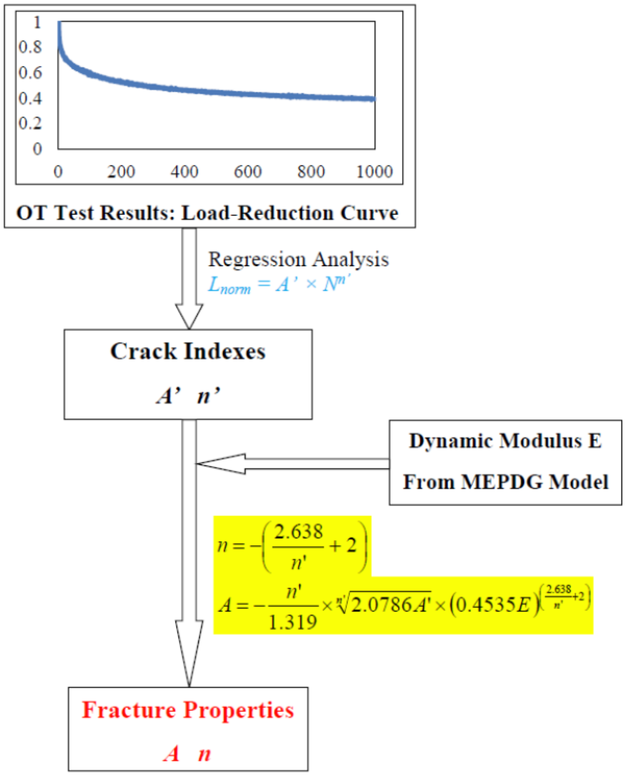


FIGURE 13 Flowchart to evaluate fracture parameter from the OT test result

TABLE 12 Summary of the Texas OT test procedure and interpretation of results from different agencies

Agency	Spec title	Sample size	Temp, °F	Loading rate	frequency	Other config.	Outcome	Spec value	Max COV
FL (*38*)	No designation yet	6in*3in*1.5in	77±1	0.025in Triangular	0.1Hz	V_a 7±1%,	Cycles to 93% load reduction	300 cycles for surface, 750 for crack resistance mix	30% for three samples
IL (*39*)	TEX-248F	6in*3in*1.5in	77±1	0.025in Triangular	0.1Hz	V_a 7±1%,	Cycles to 93% load reduction	300 cycles for surface, 750 for crack resistance mix	
NV (*40-41*)	Modified TEX-248F	6in*3in*1.5in	50±1	0.018in Triangular	0.1Hz	V_a 7±0.5 %,	Cycles to 93% load reduction	1200 cycles	
TX (*42*)	TEX-248F	6in*3in*1.5in	77±1	0.025in Triangular	0.1Hz	V_a 7±1%,	Cycles to 93% load reduction	300 cycles for surface, 750 for rich bottom layer.	30%

NJ (*43*)	NJDOT B-10	6in*3in *1.5in	77±1 Or 59	0.025in Triangular	0.1Hz	V_a 7±1%,	Cycles to 93% load reduction	With RAP: Surface course: 150 for PG64-22, 175 for PG76-22 Intermediate course: 100 for PG64-22 and 125 for PG 76-22 SMA and OGFC 300, Rich bottom layer: 700	
New Zealand	TEX-248F	6in*3in *1.5in	77±1	0.025in Triangular	0.1Hz	V_a 7±1%,	Cycles to 93% load reduction	300 cycles for surface,	30%

BEAM FATIGUE TESTING

In the beam fatigue test, cyclic strain level of 600 με was applied at 10 Hz of loading upon applying sinusoidal waveform with no rest period at 20 °C until the material failed. According to the AASHTO T 321 (*44*) test protocol, the stiffness at the 50th cycle of loading is considered the initial stiffness and the number of cycles at 50% reduction of stiffness is considered the fatigue life. The support conditions and the geometry of the sample followed the requirements of the AASHTO T 321-07 test standard. The test setup is shown in Figure 14, where a sample has been clamped for testing. The middle two clamps are loading actuator, which applies force to attain the predefined strain in the sample. Summary of beam fatigue testing of different agencies are shown in Table 13.

TABLE 13 Four-point bending test procedure and outcome of different agencies

Agency	Spec title	Sample size	Temp	Loading rate	Other config.	Outcome	Spec value	Max COV
ASTM D7460 (*45*) and AASHTO T321		380±6 L, 50±2 H, and 63±2 W	Room temp, Usually 20C	200-800με	Initial stiffness = at 50 the cycle, 10Hz frequency	Number of cycles to 50% reduction of initial area	23M with virgin and 25k with RAP at 400με Or 345M with virgin and 950K with RAP at 200με Or 100k for 1500με	Std <0.278 for three replicas

							Or 100M for the rich bottom at 800με	
SHRP (*46*)	M009	381±6.35 L, 50.8±6.35 H, 63.5±6.35 W	20±1C	Variable so that N_f> 10000	Initial stiffness = at 50 th cycle, 10Hz frequency	Number of cycles to 50% reduction of initial area		
Caltrans (*47*)	AASHTO T321	380±6 L, 50±2 H, and 63±2 W	Room temp, Usually 20C	200-800με	Initial stiffness = at 50 th cycle, 10Hz frequency	Number of cycles to 50% reduction of initial area	1. 2.7M for PG64-10 15% RAP at 200με 2. 182K for 15% RAP at 400με 3. 950k for 25% RAP 200με 4. 25k for 25% RAP 400με 5. 345M por + binder at 200με 6. 23M for 400με +binder	

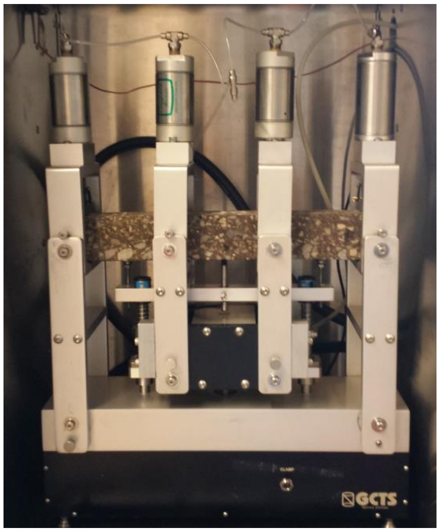


FIGURE 14 Four-point bending test setup

THERMAL STRESS RESTRAINED SPECIMEN TEST (TSRST)

Recently, TSRST has become a popular test method to evaluate the thermal fracture property of AC. TSRST is conducted by cooling an asphalt concrete specimen at a specified rate while maintaining the specimen at a constant length. A typical stress-temperature curve obtained in TSRST is divided into two parts: relaxation and nonrelaxation. The temperature at which the curve is divided into two parts is termed the transition temperature. The temperature at fracture is termed the fracture temperature, and the maximum stress is the fracture strength.

The test procedure is standardized under the AASHTO TP10 (*48*). According to this procedure, both cylindrical or prismatic samples can be used. The dimension of the prismatic sample is 40 x 40 x 160 - 60 x 60 x 250 mm whereas the dimension of the cylindrical sample is Ø 63.5 x 254 mm. Samples are restrained at both ends and are placed inside the test chamber. The sample length is kept the same, and the temperature is decreased gradually. Tensile stress develops within the sample, and when it exceeds the tensile strength, fracture occurs. The temperature at which fracture occurs as well as the maximum load generated within the sample are recorded. Figure 15 shows a TSRST test setup.

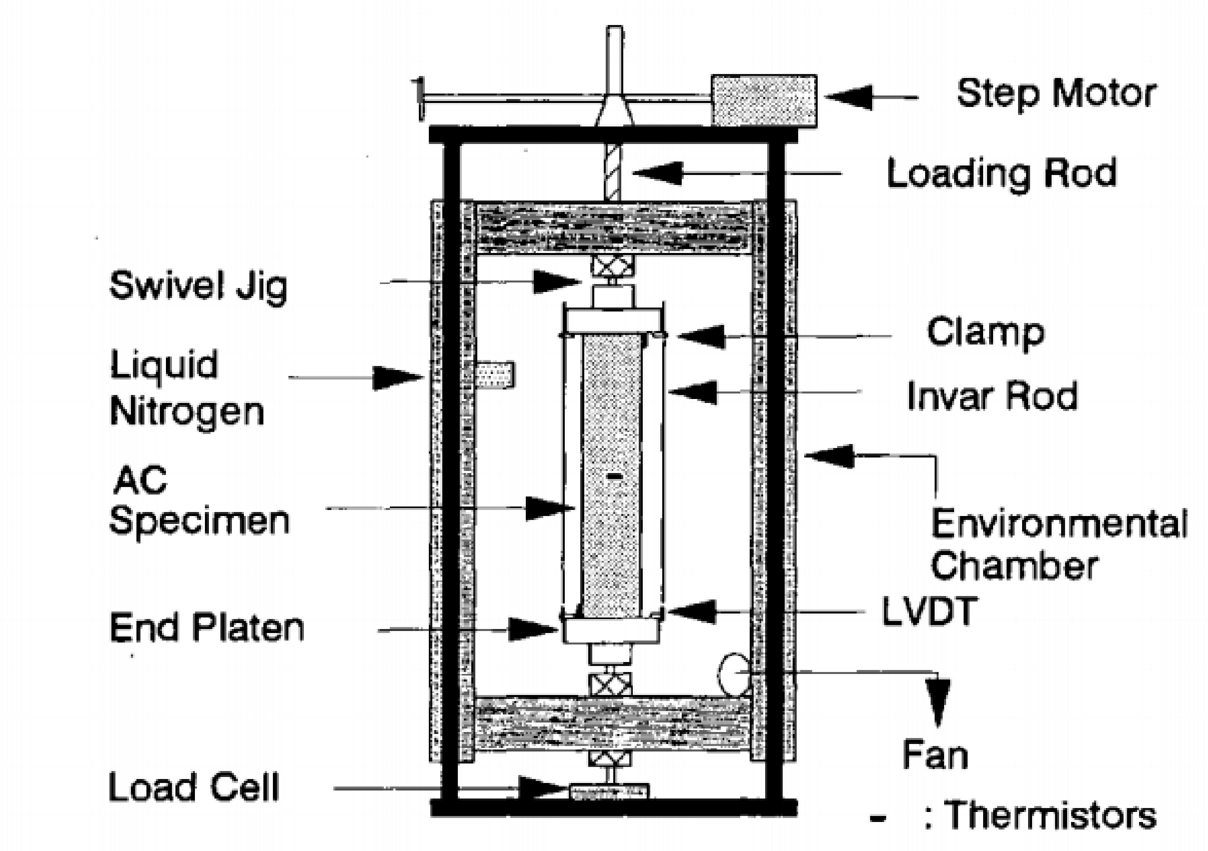


FIGURE 15 Schematic of TSRST test

The evaluation of TSRST and development of specification are still ongoing (*48*). There is no available TSRST specification yet.

SELECTION OF TESTS

Based on the literature review, the following tests were selected for different performance criteria (Table 14):

TABLE 14 Selected tests for this study

Criteria	Test
Low-temperature cracking	Disk-Shaped Compact Tension (DCT) Test
Medium temperature cracking	Semicircular (SCB) bending test
Reflection cracking	Texas Overlay Test
Rutting	Hamburg wheel tracking test (HWTT)

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DEVELOPING BMD PROCEDURE FOR NEW MEXICO

INTRODUCTION

The second BMD procedure with some modification is used for this study. It consists of the following steps:

1. Select a trial blend of aggregate.
2. Determine the optimum binder content.
3. Determine the rutting and cracking performance at optimum binder content.
4. Determine the moisture susceptibility.
5. Check whether the performance and moisture damage criteria are fulfilled.
6. Repeat all the tests at optimum + 0.5% binder content.
7. Repeat all the test at optimum + 1% binder content.
8. Select the binder content that satisfies all the performance criteria.
9. Redo the procedure with a new blend of aggregate if the performance criteria are not met with any binder content.

STEP 1: SELECTION OF AGGREGATE BLEND

Materials Collection

Materials were collected from a construction project with control number 2101771 near Artesia, NM with the help of Shawn Hammer of Fisher Sand and Gravel. Total 100 bags of aggregate, 20 bag of mix, 30 gallons of asphalt binder and 10 pounds of Versabind were collected. The 100 bags of aggregates were composed of the following sizes: 21 bags of ¾” minus, 32 bags of ½” minus, 32 bags of ¼” minus, and 15 bags of Fine RAP. A short description of the collected materials is provided below.

Mixing Procedure

Twenty bags of mix were collected. Each bag weighed about 25 pounds. The mixture was a SPIV Warm Mix Asphalt (WMA) with 15% RAP. Foam mix technology was used to mix the binder with aggregate. Versabind was used as WMA additives. Design ESALs are 3.9 million. Therefore, design gyration was selected as 100. A binder grade of PG76-22 was used. Other mix properties are shown in Table 15.

TABLE 15 Mix Design Properties of the Collected Mix

Design Parameters	Design Value	Specific Limit	Design Parameters	Design Value	Specific Limit
AC	5%	4.5 to 5.5%	Unit weight	147.7 pcf	
RAP AC	0.6%		Gmb at Ndes	2.368	
Virgin AC	4.4%		Gmm at design AC	2.467	
Air voids	4%	2.6 to 5.4%	Gse	2.655	
VMA	14.7%	13.1 to 16.3%	Density at Nini (8)	85.3%	
VFA	72.7%	68 to 75%	Density at Nmax (160)	97.5%	
Absorbed AC	0.29%		Dust proportion	1.1	0.8-1.4%
Effective AC	4.7%		Water @ mixing the versabind + aggregate		
Versabind	1%	1 to 1.4%	Tensile Strength Ratio (TSR)	98%	85%

Aggregate

Aggregate from ¾″ minus, ½″ minus, ¼″ minus, fine RAP and versabind were blended together to find out the design gradation. Table 16 summarizes the gradation of these sources.

TABLE 16 Gradation of Different Stockpiles

Sieve sizes	¾″ minus	1/2″ minus	¼″ minus	Fine RAP	Versabind
inch	Percent passing				
¾″	100	100	100	100	100
1/2″	85	100	100	98	100
3/8″	21	97	100	96	100
#4	1	25	98	64	100
#8	1	2	71	43	100
#16	1	1	41	32	100
#30	1	1	23	27	100
#50	1	1	15	23	100
#100	1	1	11	15	100
#200	0.9	1.1	9.5	9.7	100

Those stockpiles are blended at four different trials. Based on the AC, VMA, VFA, G_{mm}, and $V_{be,}$ of the trial mixes, the following blend is selected with the final gradation as shown in Table 17.

TABLE 17 Blend and Gradation of the Selected Blend

Stockpile	¾″ minus		½″ minus		¼″ minus		Fine RAP		Versabind	
% Blend	20		32		32		15		1	
Sieve Size	¾″	½″	3/8″	#4	#8	#16	#30	#50	#100	#200
Percent passing	100	97	83	50	31	20	13	10	7	6.1

Table 18 shows the properties of the final blend. All values are within the specified limit.

TABLE 18 Properties of the Final Blend

Property	Design value (pit)	RAP value	Spec limit
Flat and elongated particle	4%	3%	Max 20%
Fine aggregate angularity	47.2%		Min 45%
CA fracture faces	99%	94%	Min 85%
Sand equivalent	52%		Min 45%
Soundness loss	2%		Max 15%
LA wear	20%	26%	Max 40%
Aggregate index	9		Max 25

As the same aggregates used for project 2101771 are used in this study, the same blend as provided by Fisher Sand and Gravel NM was used. After mixing the aggregates at that proportion, the final gradation of the blend was determined in the laboratory as shown in Figure 16. If any of the design requirement was not met by this blend, the blend would be modified.

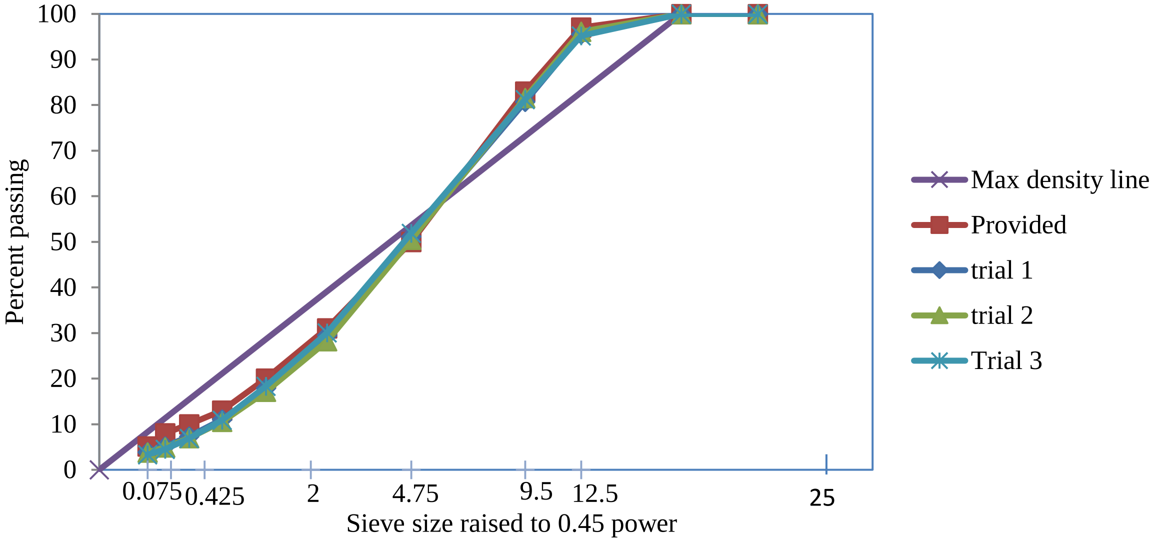


FIGURE 16 Gradation of the selected blend

STEP 2: DETERMINATION OF THE OPTIMUM BINDER CONTENT

Aggregates were taken from the different stockpile at the percentage provided in Table 17 to get the total amount of 21000 gm. They were mixed with a total of 4.5% of asphalt binder. RAP contributed 0.6% of the total binder. Therefore, 3.9% of virgin binder was added. Mixing temperature was 160 ºC. From the mix, three cylinders were compacted using the gyratory compactor. A number of gyrations, compaction temperature, and mix amounts were 100, 155 ºC and 4720 gm respectively. The rest of the mix was used to make the loose mix. The whole process was repeated for 5.0% and 5.5% binder content. The cylinders were used to evaluate bulk specific gravity of the mix (G_{mb}) and the loose mix was used to evaluate the maximum specific gravity of the mix. Bulk and maximum specific gravity were used to determine the air void at design gyration of 100. A short description to determine the air void is provided below.

Determination of Bulk Specific Gravity

The bulk specific gravity was determined using AASHTO T166 procedure. The dry weight of a specimen is determined. The specimen is then submerged under water of 25 ºC temperature for about 5 minutes. The underwater weight is determined. The sample is removed from the water and quickly surface-dried with a moist cloth and surface dry weight is determined. The bulk specific gravity is determined using Eq. (14)

$$G_{mb} = \frac{A}{B-C} * 0.997 \tag{14}$$

where, A = weight in air, gm; B = surface dry weight, gm; C = weight under water, gm; and 0.997 is the water density (gm/cc) at 25 ºC temperature.

Determination of Maximum Specific Gravity

The maximum specific gravity of loose mix was determined as per ASTM D6857: automated vacuum sealing procedure. Sample weight in air is determined. The sample is placed inside a plastic bag and vacuum sealed. The sealed bag is cut open under water, and the weight of the sample with the bag is determined underwater. The maximum specific gravity is determined using this data and GravitySuite^{TM} software. The software uses the following Eq. (15).

$$G_{mm} = \frac{B}{A+B-C-A/V_c} * 0.997 \tag{15}$$

where, B = weight of loose mix in air, gm; A = weight of the plastic bag, gm; C = submerged weight of bag with the mix, gm; V_c = Apparent specific gravity of the plastic bag.

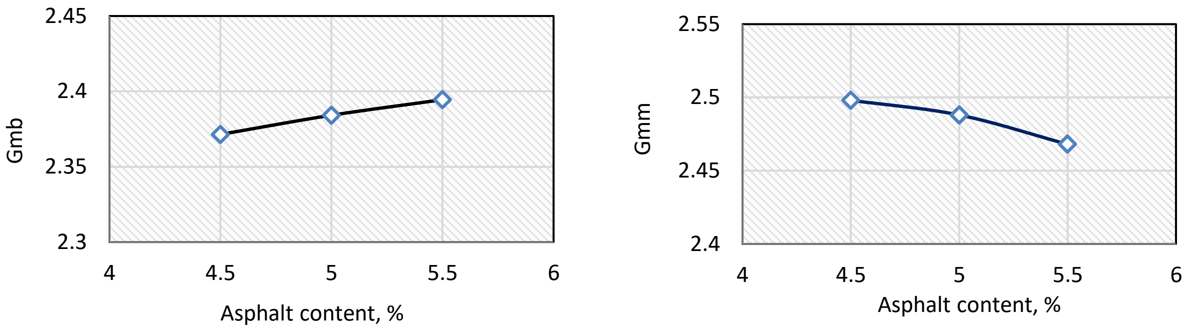
Once the bulk and maximum specific gravity are determined, air void (V_a) of the gyratory compacted specimen can be determined as:

$$V_a = \frac{G_{mm}-G_{mb}}{G_{mm}} * 100\% \tag{16}$$

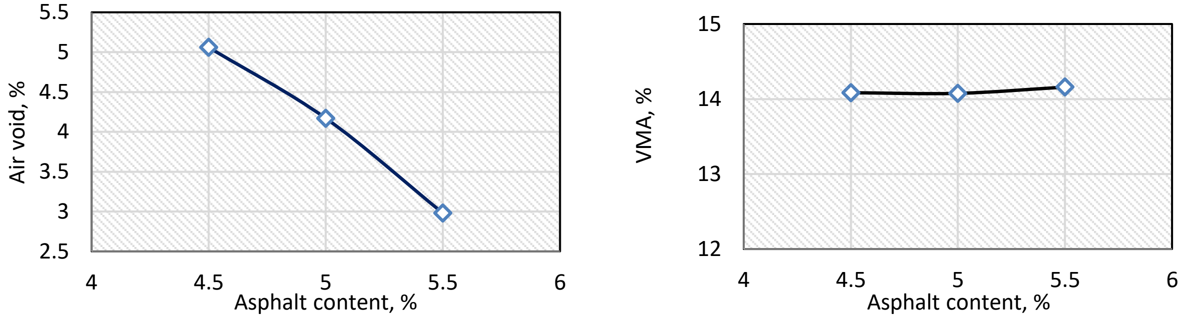
The VMA and VFA of the compacted mixes were also determined. G_{mm}, G_{mb}, V_a, VFA, and VMA for all three binder contents are summarized in Table 19 and shown in Figure 17(a) to 17(e). From Figure 17(c), optimum binder content was determined as 5.1% which is the binder content corresponding to 4% air void. The optimum binder content of the collected mix is 5.1% as observed from Figure 17(d) that VMA is within the acceptable limit for any of these binder content. VFA is lower than acceptable limit for lower binder content; however, VFA increases with the increase of binder content.

TABLE 19 Mix volumetric at different binder content

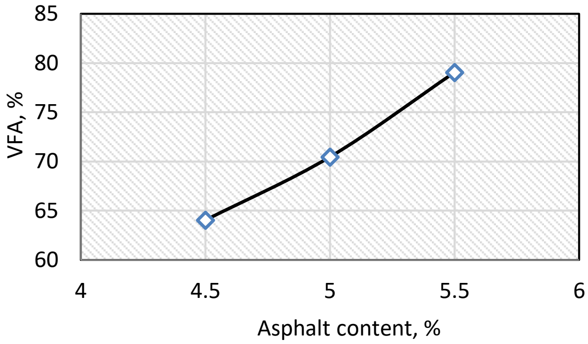
Binder content, %	G_{mb}	G_{mm}	V_a	VMA	VFA
4.5	2.371	2.498	5.1	14.1	64
5	2.384	2.488	4.2	14.1	71
5.5	2.394	2.468	3.0	14.2	79



(a) G_{mb} vs. asphalt content (b) G_{mm} vs. asphalt content



(c) Air void vs. asphalt content (d) VMA vs. asphalt content



(e) VFA vs. asphalt content

FIGURE 17 Changes in mix volumetric with asphalt content

STEP 3: PERFORMANCE TESTS FOR BMD

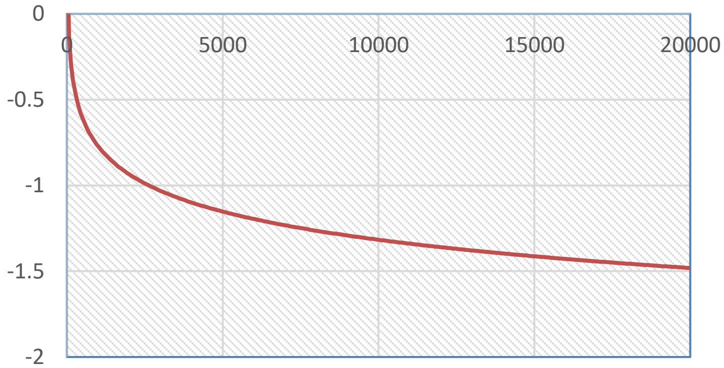
Test for Rutting Performance

HWTT Test

HWTT was first used in Germany in the mid-70s. Cylindrical or slab samples are placed in the mold. Metal wheels of 158 lbs weight roll over the sample for 20000 cycles. The rut depth is measured at different cycle intervals. After several cycles, the rut depth increases suddenly, which is a measure of moisture damage susceptibility. Several states have their specification for HWTT. Figure 18(a) shows a prepared sample, and Figure 18(b) shows the test in progress. Figure 18(c) shows the test results. It is observed that rutting corresponding to 10000, 15000 and 20000 cycles is 1.3 mm, 1.4 mm and 1.5 mm respectively. All values are within the specified limit of New Mexico (3.5, 4 and 4.5 mm).

(a) Prepared samples

(b) HWTT in progress



(c) HWTT test results

FIGURE 18 Hamburg wheel tracker test

Test for Cracking Performance

SCB Test

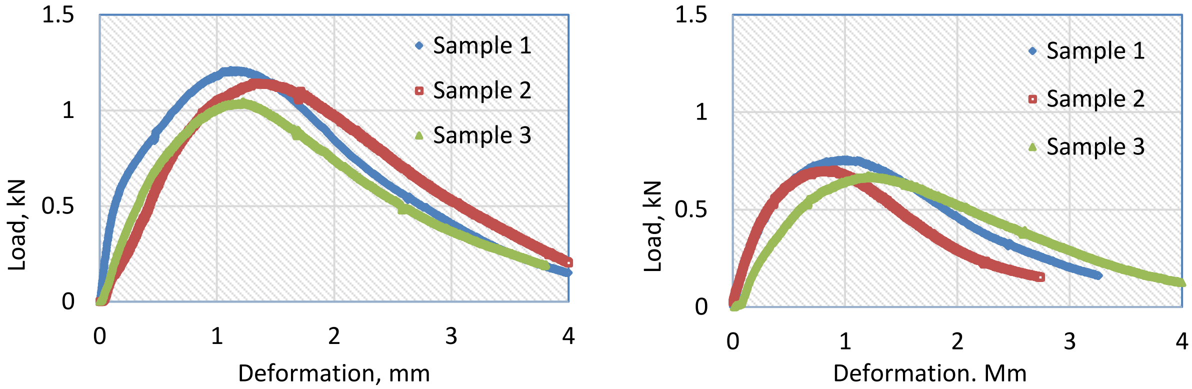
Four cylinders of 150 mm height are compacted using the gyratory compactor using 6080 grams of the mix for each. The quantity came from the weight volume relationship. From each cylinder, 4 SCB samples were extracted from each cylinder. The standard sample is 57 mm thick. To determine *J-integral*, strain energy for three different notch depths are required. Three samples are required for each notch depth. Therefore, a total of 9 samples were tested. The sample was loaded at 0.5 mm/min rate. The sample preparation and test set-up are shown in Figure 19.

(a) A prepared sample (b) Test set-up

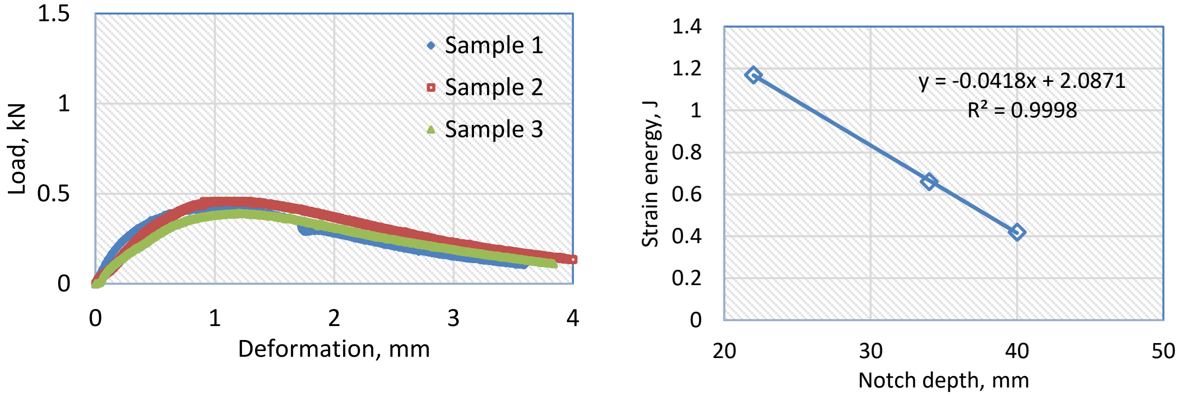
FIGURE 19 SCB test procedure

Load and load line deformation data were collected for each sample and plotted. The area under the curve until the peak load is termed as critical strain energy or strain energy. Stain energy for three samples of the same notch depth was evaluated, and the average value and COV of the three samples are determined. A similar approach was applied to two other notch depths.

The load-displacement test results for three different notch depth are shown in Figure 20(a) to (c). The outcome from those three plots is shown in Table 20. Figure 25(d) shows the strain energy vs. notch depth plot. The value of J-integral is obtained as 0.75 kJ/m^2 which is higher than the minimum value specified. The COV is obtained as 4%, 8% and 7% for 22 mm, 34 mm and 40 mm notch depths respectively.



(a) 22 mm notch depth (b) 34 mm notch depth



(c) 40 mm notch depth (d) Strain energy vs. notch depth

FIGURE 20 Determination of *J-integral*

TABLE 20 Data extracted from the P-D plot

Sample ID	Notch depth, mm	Peak load, kN	Strain energy, *J*	Average, J	COV, %
1	22	1.21	1.24	1.17	3.85
2		1.15	1.15		
3		1.05	1.13		
1	34	0.76	0.72	0.66	7.72
2		0.71	0.59		
3		0.68	0.66		
1	40	0.43	0.40	0.42	6.67
2		0.46	0.46		
3		0.40	0.40		

DCT test

Three cylinders of 80 mm height were compacted using a gyratory compactor using 3240 grams of the mix. From each cylinder, one 50 mm thick DCT sample was extracted. The sample was trimmed, and the notch was cut using the standard template to obtain a final specimen as shown in Figure 5(a). Samples were tested at -12 ºC at a rate of 1 mm/min. Load and crack mouth opening displacement (CMOD) are recorded and plotted.

The test result for the laboratory mix at optimum binder content is shown in Figure 21. The area under the curves was determined. The area is divided by the ligament length and thickness of the sample to determine the fracture energy. The fracture energy obtained for this mix is 49 $J/\ m^2$ which is very low compared to the standard value of 450 J/m^2.

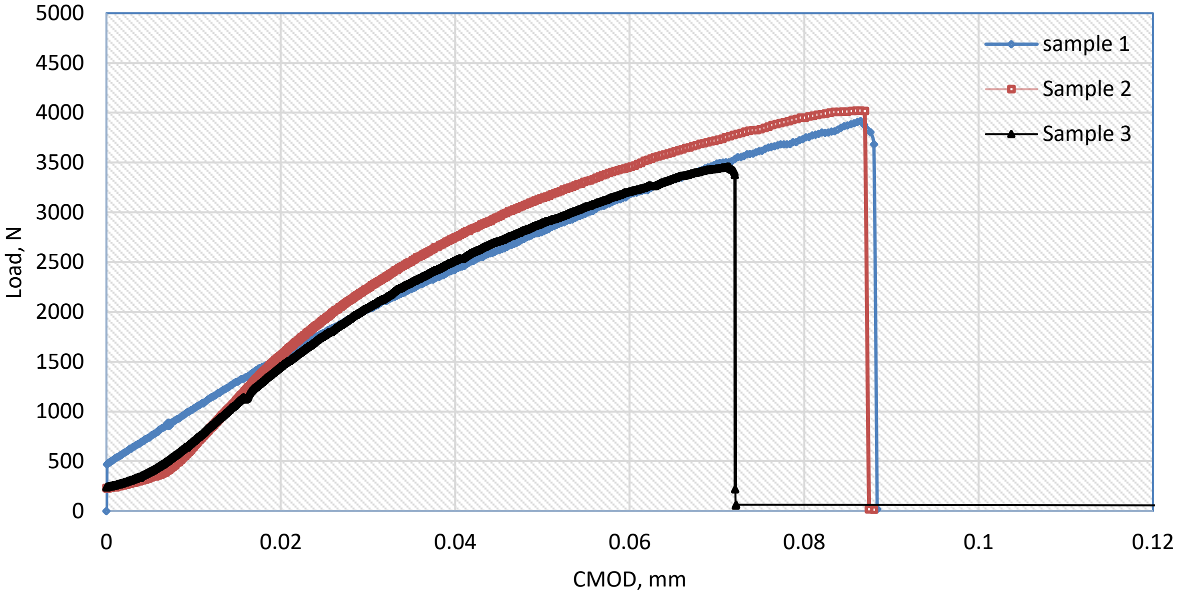


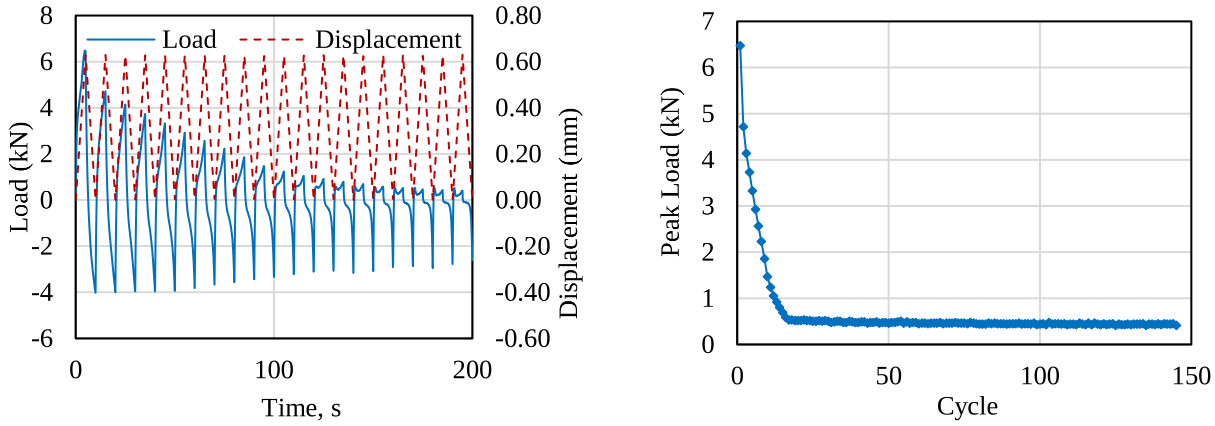
FIGURE 21 Load vs. CMOD for laboratory mix at optimum binder content

Texas Overlay Test

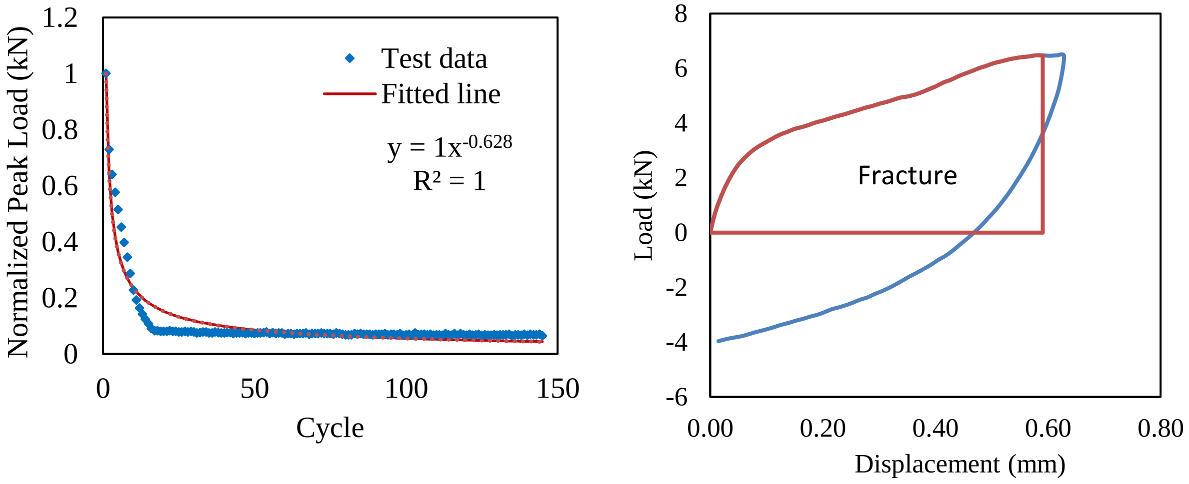
Figure 22(a) and (b) shows the recently acquired Texas OT device and sample mounting respectively. Testing was performed at 77 ± 1°F (25 ± 0.5°C) temperature with a displacement amplitude of 0.025 in. at a frequency of 10 Hz. Load deformation data was collected and plotted as shown in Figure 23(a). Crack initiated at the first cycle of loading. Load amplitude got lower over time due to crack initiation and propagation. Figure 23(b) shows the reduction of load with the number of cycles. The test was stopped when the load was reduced by 93%. The power of the exponential fit of this graph is known as β – parameter, which is -0.628 as shown in Figure 23(c). Fracture energy was calculated by considering area under the load-deformation plot for the first cycle normalized with respect to thickness and width of the sample. The load-deformation plot for the first cycle is shown in Figure 23(d). The fracture energy was obtained as 943 J/m^2. As only one test was successful, no conclusion is drawn for this test.

(a) Texas OT device (b) Sample mounting

FIGURE 22 OT device and sample mounting



(a) Load or displacement over time (b) Peak load reduction over time



(c) Calculation of β – parameter (d) Area considered to calculate critical fracture energy

FIGURE 23 Interpretation of OT data

STEP 4: EVALUATE MIX DESIGN REQUIREMENTS AT OPTIMUM BINDER CONTENT

Cylinders were compacted, and the loose mix was prepared to determine G_{mb} and G_{mm} of mix prepared using 5.1% asphalt binder and the same procedure described in step 2. At the same time, the same mix was compacted by 8 and 160 gyrations to evaluate the density at N_{ini} and N_{max}. The density at N_{ini} was obtained as 87% which was lower than the design criteria of 89%. The density at N_{max} was 96.9%, which is lower than 98%.

STEP 5: DETERMINATION OF MOISTURE SUSCEPTIBILITY

IDT Test

Moisture susceptibility was determined using AASHTO T283 procedure. AASHTO T 283. In the AASHTO T 283 method, an asphalt core is saturated using a Vibro-Deaerator device that applies vibration and suction simultaneously. After saturation, it is wrapped inside plastic papers and placed inside a moist ziplock bag. The sample is then placed in a refrigerator for 16 hours at -23 °C (0 °F) for freezing. The sample is then thawed in a water bath at a temperature of 60 °C (140 °F) for 24 hours followed by two more hours of conditioning at 25 °C (77 °F). Thus, in AASHTO T 283 conditioning process, water that enters inside the sample during saturation increases in volume during freezing. The increased volume of ice causes damage to the samples. Thawing by hot water for 24 hours also contributes to the damage. For dry conditioning, the dry sample is placed inside a Ziploc bag and placed under water at 25 °C (77°F) for two hours.

For the TSR test, a dry or wet conditioned sample is loaded to fail diametrically. The load is applied at 2 in./min rate, and the peak value of the load was recorded. Then, the indirect tensile strength (*IDT*) is calculated using Eq. (17) below,

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

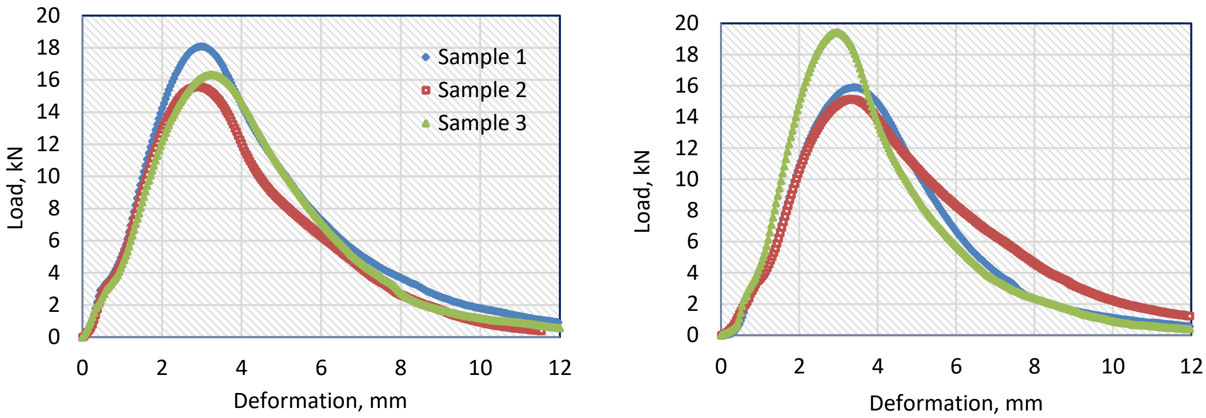
where IDT = indirect tensile strength (kPa);

P = peak force needed to break the sample diagonally, recorded from the compression testing device (pound); D = diameter of the sample (inch); t = length of the sample (inch); TSR is determined from Eq. (18)

$$TSR = \frac{IDT_{wet}}{IDT_{dry}} \tag{18}$$

where TSR = tensile strength ratio; IDT_{wet} = Average *IDT* of moist conditioned samples; IDT_{dry}= Average *IDT* of dry conditioned samples.

Figure 24(a) shows the indirect tensile strength test results for dry mix and Figure 24(b) shows the results for wet conditioned samples. The average and COV of the three dry IDT are 244 psi and 8%. The values for moist samples are 246 psi and 13%. The TSR value obtained is 1.0, which is greater than the minimum specified limit of 0.85.



(a) IDT test results for dry samples (b) IDT test results for wet conditioned samples

FIGURE 24 Test results for indirect tensile strength

SUMMARY OF THE TEST RESULTS

All performance tests were performed for laboratory designed mix at optimum binder content. The test results are summarized in Table 21. It is observed that the mix satisfied the rutting, moisture damage, medium temperature cracking, however, didn't satisfy the low temperature cracking resistance. Therefore, binder content needs to be raised by 0.5% and all the performance tests need to be conducted.

TABLE 21 Summary of test results

Tests	TSR	SCB, kJ/m^2	DCT, J/m^2	HWTT, mm
Results	1.0	0.75	49	1.3, 1.4, 1.5
Specified limit	0.85 (NMDOT)	0.6	400	3.5, 4, 4.5

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CONCLUSIONS

CONCLUSIONS

The following conclusions can be drawn from this study:

a) Based on the literature, a step-by-step procedure to perform a balanced mix design was evaluated. There are three approaches to perform a balanced mix design. The performance-modified volumetric design option was selected for this study as it requires fewer trials, materials, and it is straight forward.
b) The following tests were selected for developing BMD for NMDOT: SCB test for intermediate temperature cracking, DCT for low temperature cracking, OT test for reflective cracking, and HWTT for rutting. Currently, NMDOT has specifications, for HWTT. However, there is no specification for cracking. This study came up with limiting values of different cracking based on literature.
c) Aggregate and asphalt binder were collected from a project. A mixture was designed using the Superpave procedure. All the performance tests were performed on the mix at optimum binder content. The SCB, HWTT, and TSR results satisfied the limiting values. However, DCT results were not satisfactory.
d) An overlay tester was acquired, installed and training was provided. Limited tests were conducted to gain experience and to have confidence in the test data.

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APPENDIX A: PERFORMANCE TEST RESULTS OF THE FIELD MIX

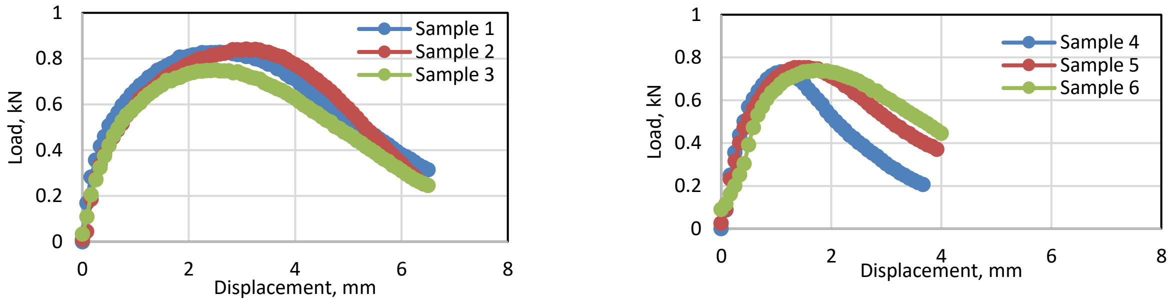
INTRODUCTION

In addition to designing an HMA mix in the laboratory using Superpave procedure, a WMA mix from the field was collected that used the same materials and aggregate blending. The only difference is that the laboratory mix was HMA whereas the collected mix was WMA. Performance of the collected mix was evaluated to have an idea about the performance of a field mix for New Mexico and to discuss how the results deviate from the HMA.

SCB TEST RESULTS

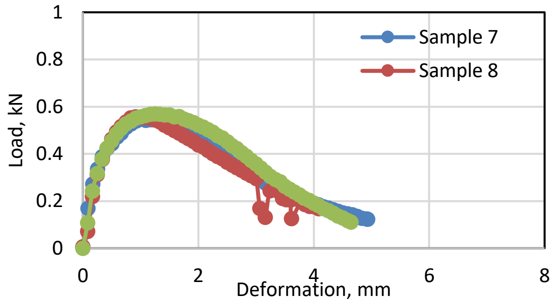
Four cylinders of 150 mm height were compacted using the gyratory compactor using 6080 grams of the mix for each. The quantity came from the weight volume relationship. From each cylinder, 4 SCB samples were extracted from each cylinder. The standard sample was 57 mm thick. To determine *J-integral*, strain energy for three different notch depths were required. Three samples were required for each notch depth. Therefore, a total of 9 samples were tested.

Load and load line deformation data were collected for each sample and plotted. The area under the curve until the peak load was termed as critical strain energy or strain energy. Strain energy for three samples of the same notch depth were evaluated, and the average value and COV of the three samples were determined. A similar approach was applied to two other notch depths. Figure A-1(a) to (c) shows the plots for all nine samples and Table A-1 shows the outcome from those plots. It is observed that the COV values for three notch depths are 12%, 16%, and 18%.



(a) Load deformation plot for 18 mm notch

(b) Load deformation plot for 33 mm notch



(c) Load-deformation plot for 39 mm notch

FIGURE A- 1 Load vs. deformation plot for three different notch depths

TABLE A-1 Data extracted from load-deformation plot

Sample ID	Notch depth, mm	Peak load, kN	Strain energy per unit thickness, kJ/m	Average, kJ/m	COV, %
1	17.5	0.83	0.031	0.029	12
2		0.84	0.033		
3		0.75	0.025		
4	33.9	0.73	0.011	0.014	16
5		0.75	0.016		
6		0.74	0.015		
7	38.2	0.55	0.007	0.008	18
8		0.56	0.006		
9		0.57	0.009		

Once strain energy for three notch depths was evaluated, they were plotted against the notch depth as shown in Figure A-2. Then *J-integral* was evaluated using Eq. (6). The *J-integral* value was obtained as 1.02 kJ/m^2 which is much higher than the limiting value of 0.6 kJ/m^2. This indicates that the base mix has higher cracking resistance at medium temperature.

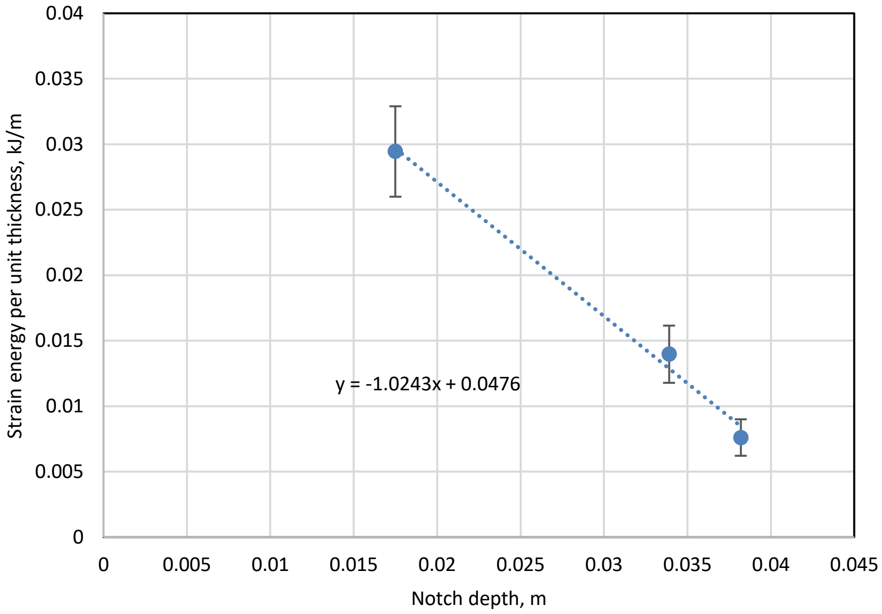


FIGURE A-2 Strain energy vs. notch depth

DCT TEST RESULTS

Three cylinders of 80 mm height were compacted by a gyratory compactor using 3240 grams of the mix. From each cylinder, one 50 mm thick DCT sample was extracted. The sample was trimmed, and the notch was cut using the standard template to obtain a final specimen as shown in Figure 5(a). Samples were tested at -12 ºC at a rate of 1 mm/min. Load and CMOD were recorded and plotted as shown in Figure A-3. The area under the curve was determined. Fracture energy is evaluated using Eq. (13). Fracture energy of *0.21 kJ/m*^{2} was obtained (with a COV of 17%), which is very low compared to the standard value of other states (0.45 kJ/m^2 for Louisiana).

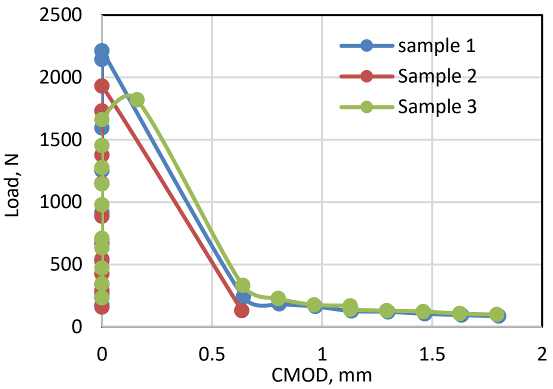
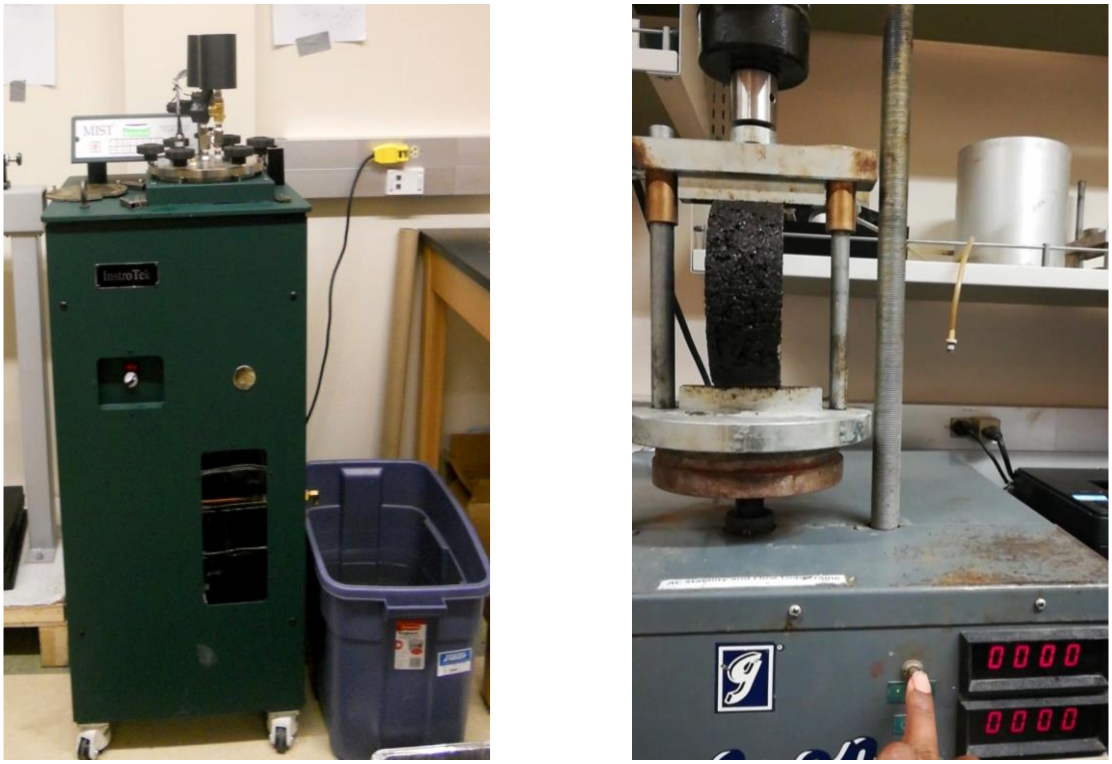


FIGURE A-3 Load vs. COMOD plot

TSR TEST RESULTS

This test is a part of the performance test, however a coherent part of the volumetric mix design. Therefore, it is included here instead of the performance section.

TSR was determined using Moisture Induced Sensitivity Test (MIST). The test set up for MIST is shown in Figure A1-4(a). In this method, a cylindrical sample is placed inside the MIST chamber filled with water. A bladder inside the watertight chamber is used to increase and decrease the chamber pressure. In this study, the chamber temperature was set at 60 °C (140°F) with a chamber pressure of 40 psi. A total of 3500 cycles of pressure increase and release were made. As the number of cycles increases, the air inside the sample is replaced by water. At certain intervals, the air bubbles are released from the top of the chamber lid. Water inside the sample applies cyclic pore pressure on the sample and causes damage. When the load cycles are complete, the sample is removed and placed in room temperature water for 2 hours.



(a) MIST test set-up (b) IDT test set-up

FIGURE A-4 MIST conditioning and IDT test

For the TSR test, a dry or wet conditioned sample is loaded to fail diametrically as shown in Figure A-4(b). The load is applied at 2 in/min rate, and the peak value of the load is recorded. Then, the indirect tensile strength is calculated using Eq. (A-1) below,

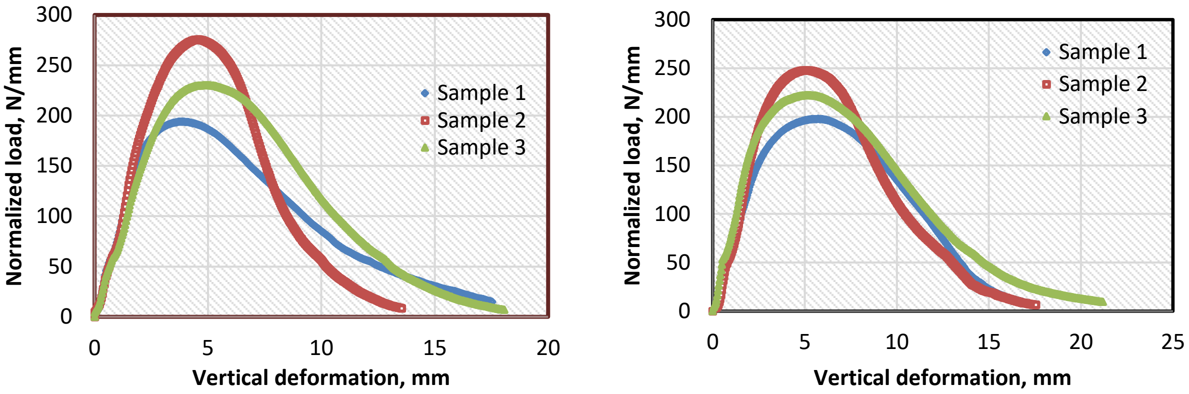
$$IDT = \frac{2P}{\pi D t} \qquad \text{(A-1)}$$

where *IDT* = indirect tensile strength (psi); *P* = peak force needed to break the sample diagonally, recorded from the compression testing device (pound); *D* = diameter of the sample (inch); *t* = length of the sample (inch); TSR is determined from Eq. (A-2)

$$TSR = \frac{IDT_{wet}}{IDT_{dry}} \qquad \text{(A-2)}$$

where *TSR* = tensile strength ratio; IDT_{wet} = Average *IDT* of moist conditioned samples; IDT_{dry} = Average *IDT* of dry conditioned samples.

The test results for dry and wet conditioned samples are shown in Figure A-5 (a) -(b). The resulting TSR is obtained as 0.95, which is higher than the state specified limit of 0.85.

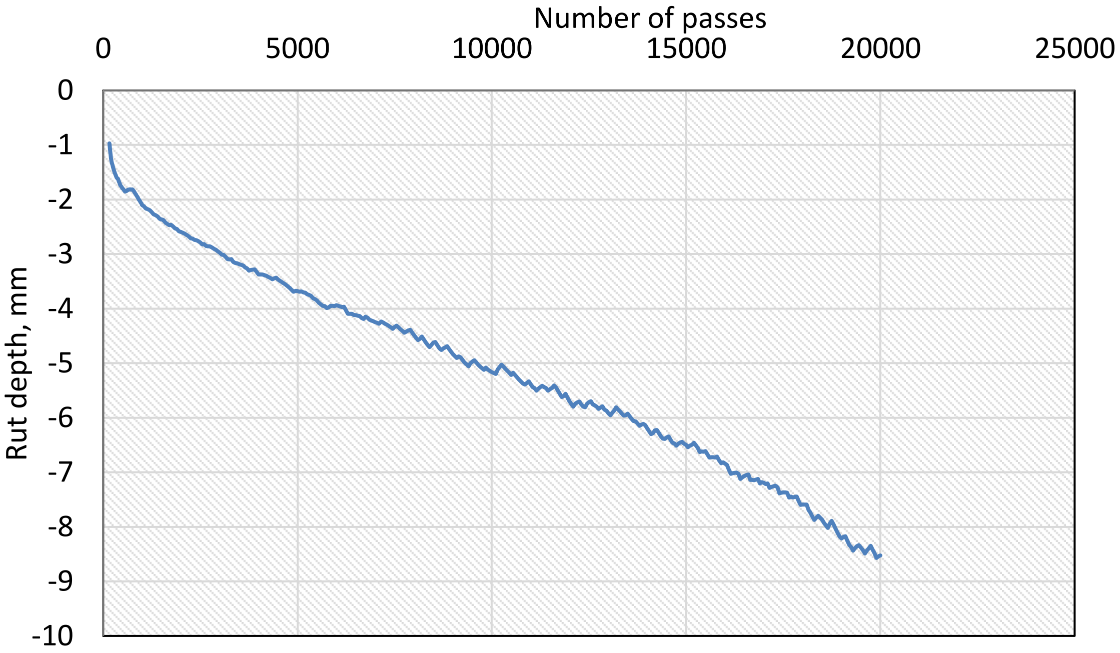


(a) Dry samples (b) Wet conditioned samples

FIGURE A-5 Load-deformation plot from the IDT test

HWTT RESULTS

Figure A-6 shows the test results. It is observed that the rutting corresponding to 10000, 15000 and 20000 cycles are 5 mm, 6.5 mm and 8.5 mm respectively. All values are higher than the specification value of New Mexico (3.5, 4 and 4.5 mm).



(c) HWTT test results

FIGURE A-6 HWTT test

SUMMARY OF TEST RESULTS

Table A-2 summarizes the results for the field WMA mix and compares with the laboratory HMA mix and specified limits. It is observed that the SCB and DCT results are higher for WMA than

HMA; DCT for WMA is below the minimum allowable limit. The rutting criteria are met for HMA but not for WMA.

Table A-2 Comparison of performance tests for different mixes

Tests	TSR	SCB, kJ/m^2	DCT, J/m^2	HWTT, mm
WMA Results	0.98	1.02	**210**	**5, 6.5, 8.5**
HMA Results	1.0	0.75	**49**	1.3, 1.4, 1.5
Specified limit	0.85 (NMDOT)	0.6	400	3.5, 4, 4.5

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