Development of Guidelines to Minimize Moisture Damage in HMA with PennDOT District 1 Local Aggregates

FINAL REPORT

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The sources delivering quality aggregate for use in hot-mix asphalt concrete have been depleting in PennDOT District 1. There is currently a shortage of quality Type A aggregate in this district. The local aggregates don't meet the required criteria on soundness or absorption level, or both. Furthermore, stripping and moisture damage in these gravel aggregates are of concern even though most of these aggregates exhibit excellent skid resistance and durability. A research project was initiated to evaluate performance of hot-mix asphalt concrete using District 1-0 aggregates, specifically in regard to stripping and moisture damage. Research included modifications that could be applied to improve performance of such aggregates in hot-mix asphalt. Four Type C aggregates and one Type A aggregate, all sources located in District 1-0, were selected for evaluation. Mixes were prepared as control, with liquid anti-stripping agent, with lime, and with a gravel-limestone blend for the #8 material at equal proportions. The tests included PennDOT's modified version of AASHTO Test Method T283 (Tensile Strength Ratio), Model Mobile Load Simulator, 3rd Scale (MMLS3), and dynamic modulus after repeated freeze-thaw cycles. Overall, it was concluded that two of the five aggregate sources could pass the requirement on moisture damage resistance based on the PennDOT version of the AASHTO T283 test method. It was also found that the specific liquid anti-stripping agent used with these mixes improved the moisture damage resistance significantly. The study indicated improvement of moisture damage resistance using the limestone-gravel blend to a much lesser degree compared to the improvement gained through the liquid anti-stripping agent. Testing with the MMLS3 provided valuable information. Only three control mixes were included in this part of the study, and testing was conducted under both dry and wet conditions. A higher rutting level was found in wet tested specimens compared to dry specimens, even though the significance of the moisture impact on rutting level has yet to be determined. In summary, it seems that use of the specific liquid anti-stripping agent with the mixes in this study causes significant improvement in performance of mixes using these local aggregates.</td></tr>
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EXECUTIVE SUMMARY

The sources delivering quality aggregate for use in hot-mix asphalt concrete have been depleting in Pennsylvania Department of Transportation (PennDOT) District 1. There is currently a shortage of quality Type A aggregate in this district. Out of over 30 sources of sand and gravel operation in the district, only one source is able to meet the current bituminous criteria for #8 aggregate needed for 9.5-mm asphalt mixes. The local aggregates don't meet the required criteria on soundness or absorption level, or both. Furthermore, stripping and moisture damage in these gravel aggregates are of concern, even though most of these aggregates exhibit excellent skid resistance and durability. As a result, aggregates have been hauled into the district from neighboring districts or states, resulting in an increased cost of the material and the constructed pavement.

A research project was initiated by PennDOT, in cooperation with the Mid-Atlantic Universities Transportation Center (MAUTC), to evaluate the performance of hot-mix asphalt concrete using District 1-0 aggregates, specifically in regard to stripping and moisture damage. Research included modifications that could be applied to improve performance of such aggregates in hot-mix asphalt. Five aggregate sources located in District 1-0 were selected for evaluation. Four Type C aggregates and one Type A aggregate were included in the study. Furthermore, through coordination with PennDOT, a PG 64-22 asphalt binder was selected to be used with all the mixtures.

An experimental design was developed to prepare specimens and conduct required tests. Mixes were prepared as control, with liquid antistripping agent, with lime, and with a gravel-limestone blend for the #8 material at equal proportions. The tests included the PennDOT-modified version of AASHTO Test Method T 283 (Tensile Strength Ratio), Model Mobile Load Simulator 3rd Scale (MMLS3), and dynamic modulus after repeated freeze-thaw cycles.

Overall, it was concluded that two of the five aggregate sources could pass the requirement on moisture damage resistance based on the PennDOT version of the AASHTO T283 test method without any modifier. It was also found that the specific liquid anti-stripping agent used with these mixes improved the moisture damage resistance significantly. In summary, for two of the failing aggregate sources using LAS increased TSR to values above 0.80, making the mix a passing mix in regard to moisture damage. The study indicated that the moisture damage resistance was also improved through the use of the limestone-gravel blend, but to a much lesser degree compared to the improvement gained through the liquid antistripping agent. The small impact of the limestone-

gravel blend may be attributed to the small amount of limestone in the total aggregate blend (23%). Except for one mix, using hydrated lime did not improve the tensile strength ratio of the mixes. It is well known that use of hydrated lime with siliceous gravel aggregates, in general, results in improvement of moisture damage resistance. The reasons why such behavior was not observed for the mixes used in this study is unknown at this time. The poor performance of lime treated mixes of this study could have come from the impact of the interaction between the fine material in the mix and the lime as the same fine material was used for all mixes. However, further investigation is needed to truly identify why adding lime did not improve moisture damage resistance.

Testing with the MMLS3 provided valuable information. Only three control mixes were included in this part of the study, and testing was conducted under both dry and wet conditions. Total rutting was limited to 2 to 3.5 mm for dry specimens and 2.5 to 4 mm for wet specimens. It is the authors' conclusion that moisture influenced the level of permanent deformation observed in the MMLS3 testing, even though the magnitude of this impact is not clear from the data. Further investigation is needed to determine how the observed performance in MMLS3 relates with expected field performance. It is, however, clear that based on MMLS3 data available from other sources, the rutting levels observed in our testing are not excessive and are within the range of properly designed mixes tested in the MMLS3 under other projects.

CHAPTER 1
INTRODUCTION

Aggregate constitutes almost 85 percent of hot-mix asphalt concrete by volume, and it plays a major role in performance of the pavement. The Superpave system brought significant awareness to the asphalt paving industry regarding the importance of using quality aggregate in highway construction. The new specifications on aggregate quality became more stringent. More emphasis was placed on aggregate angularity, resulting in further increases in the usage of crushed aggregates. Aggregates to be used in asphalt concrete should satisfy strict specifications in regard to shape, toughness, soundness, absorption, cleanness, crushed faces, and gradation. Unfortunately, sources of high-quality aggregates for highway construction are not evenly distributed within different states and even within a state. This has forced transportation of this material for long distances to deliver quality aggregate at construction sites, resulting in increased cost. This research was conducted to evaluate the quality of non-conforming aggregates in one of the regions within the Commonwealth of Pennsylvania facing problems with a shortage of aggregates satisfying specification requirements.

BACKGROUND

Within the last several years, the sources delivering quality aggregate for use in hot-mix asphalt concrete have been depleting in PennDOT District 1. There is currently a shortage of quality Type A aggregate. Limited sources of gravel are available to deliver AASHTO #8 material. Furthermore, stripping and moisture damage in these gravel aggregates are of concern, even though most of these aggregates exhibit excellent skid resistance and durability. As a result, aggregates have been hauled into the district from neighboring districts or states. Transport has been conducted using trucks, trains, and boats, in some cases for a distance of well over 100 miles. Obviously, this approach results in an increased cost of the material and therefore, increased cost of the final product, the constructed pavement. Approximately 30 to 35 sources of sand and gravel operation exist in the district. Unfortunately, only one source is qualified to meet the current bituminous criteria for #8 aggregate needed for 9.5-mm asphalt mixes. Even this source will probably be depleted in approximately 5 years.

Most of the local aggregates satisfy specification requirements for Type A aggregate except requirements for sodium sulfate soundness and water absorption. Therefore, PennDOT, in cooperation with the Mid-Atlantic Universities Transportation Center (MAUTC), initiated a research project to evaluate the performance of hot-mix asphalt concrete using these aggregates, specifically in regard to stripping and moisture damage. The results of this research project, referred to as Work Order 11, are expected to assist PennDOT District 1-0 in deciding an acceptable level for using local aggregates.

OBJECTIVES

The objectives of this research project can be summarized as follows:

- Evaluate the performance of District 1-0 aggregates in hot-mix asphalt concrete;
- Evaluate modifications that could be applied to improve performance of such aggregates in hot-mix asphalt; and
- Provide guidance on usage of such aggregates in hot-mix asphalt.

RESEARCH APPROACH AND SCOPE OF WORK

Through coordination with the PennDOT technical manager for this research project, five aggregate sources located in District 1-0 were selected for evaluation. Four Type C aggregates and one Type A aggregate were included in the study. Selection was based on the results from the sodium sulfate soundness test. These sources of aggregates exhibited various levels of loss based on the sodium sulfate soundness test. Furthermore, through coordination with PennDOT, a PG 64-22 asphalt binder was selected to be used with all the mixtures.

An experimental design was developed to prepare specimens and conduct required tests. Mixes were prepared as control, with liquid antistripping agent, with lime, and with a gravel-limestone blend. The tests included the PennDOT-modified version of AASHTO Test Method T 283 (Tensile Strength Ratio), Model Mobile Load Simulator 3rd Scale (MMLS3), and dynamic modulus after repeated freeze-thaw cycles. Chapter two of this report covers materials and laboratory testing. Analysis, interpretation, and findings are discussed in chapter three. The final chapter presents a summary and conclusions.

CHAPTER 2
LABORATORY INVESTIGATION

The research work conducted under this project was mainly focused on laboratory testing and evaluation of asphalt concrete mixes produced using District 1-0 local aggregates. This chapter provides an explanation of materials used in the study and the testing program used to evaluate the produced mixes.

MATERIALS

Aggregate Sources

Selection of aggregate sources was conducted through coordination with the PennDOT technical director for the project. The siliceous gravel aggregates used for this research were obtained from five different sources in District 1-0 (Table 1). The study also included a limestone source from a neighboring district. All aggregates were AASHTO designation #8. Four of the five sources were classified as Type C aggregates according to PennDOT Specification 408, section 703, course aggregate quality requirements (Table 2). One source was classified as a Type A aggregate. The fine #3 aggregate used in the study was classified as Type B, according to PennDOT specification (Table 3). This B-3 material was obtained from only one source (Table 1) and was blended with the #8 aggregate from other sources discussed above to deliver the asphalt mixes needed for the study. In addition, hydrated lime was also obtained to be utilized as an anti-stripping additive with aggregates for part of the study.

Table 1 Materials Used for This Research

Producer	Location	Penn State Code	Material Type	Sodium Sulfate, %Loss	Absorption, %	1-Face Crushed, %	2-Face Crushed, %
Troy Sand & Gravel	Waterford, Pa	TRC	C, #8, Gravel	14	3	95	90
Lakeland Sand & Gravel	Hartstown, Pa	LLC	C, #8, Gravel	20	4	92	92
Hasbrouck Sand &Gravel	Hydetown, Pa	HBC	C, #8, Gravel	14	3.2	95	87
IA Construction	Garland, Pa	IAC	C, #8, Gravel	12	3.4	91	87
Conneaut Lake Sand &Gravel	Conneaut Lake, Pa	CLA	A, #8, Gravel	8	2.1	95	90
Allegheny Mineral Corp.	Harrisville, Pa	AMA	A, #8, Limestone	3	0.65	100	100
Troy Sand &Gravel	Waterford, Pa	TR-B3	Fine Agg. - B3				
Allegheny Mineral	Harrisville, Pa	-----	Lime				
United Refining Co.	Warren, Pa	-----	PG 64-22 PG 64-22 w/anti-strip				
AKZO-NOBLE		-----	Anti-strip				

Table 2 Coarse Aggregate Quality Requirements (PennDOT Spec 408, Section 703)

	Type A	**Type B**	**Type C**
Soundness, max. %	10	12	20
Abrasion, max. %	45	45	55
Thin and Elongated Pieces, max. %	15	20	—
Material Finer Than 75 μm (No. 200) Sieve, max. %	*	*	10
Crushed Fragments, min. %	55	55	50
Compact Density (Unit Weight), min. kg/m^3 (lb/cu ft)	1100 (70)	1100 (70)	1100 (70)
Deleterious Shale, max. %	2	2	10
Clay Lumps, max. %	0.25	0.25	3
Friable Particles, max. % (excluding shale)	1.0	1.0	—
Coal or Coke, max. %	1	1	5
Glassy Particles, max. %	4 or 10	4 or 10	—
Iron, max. %	3	3	3
Absorption, max. %	3.0	3.5	—
Total of Deleterious Shale, Clay Lumps, Friable Particles, Coal, or Coke Allowed, max. %	2	2	15

Table 3 Fine Aggregate Grading and Quality Requirements (PennDOT Spec 408, Section 703)

	Cement Concrete Sand	Bituminous Concrete Sand Type B				Mortar Sand
Sieve Size	**Type A**	**#1**	**#2**	**#3**	**Filler**	**Type C**
9.5 mm (3/8-inch)	100	100	---	100	---	---
4.75 mm (No. 4)	95-100	95-100	100	80-100	---	100
2.36 mm (No. 8)	70-100	70-100	95-100	65-100	---	95-100
1.18 mm (No. 16)	45-85	40-80	85-100	40-80	---	---
600 µm (No. 30)	25-65	20-65	65-90	20-65	100	---
300 µm (No. 50)	10-30	7-40	30-60	7-40	95-100	---
150 µm (No. 100)	0-10	2-20	5-25	2-20	90-100	0-25
75 µm (No. 200)	---	0-10	0-5	0-10	70-100	0-10
Material Finer than 75 µm (No. 200) Sieve, max. percent passing	3	---	---	---	---	---
Strength Ratio, min. percent	95	---	---	---	---	---
Soundness Test, max. loss percent	10	15	15	15	---	10
Fineness Modulus	2.30-3.15	---	---	---	---	1.6-2.5

It can be seen from Figure 1 that three of the five aggregate sources do not satisfy the requirements of Type A aggregate on absorption, and one aggregate barely satisfies the criteria for a Type B aggregate. From Figure 2 it is also noticed that four of the five aggregate sources do not pass Type A requirements on sodium sulfate soundness and three of these four do not even meet the sodium sulfate soundness loss for a Type B aggregate. Considering combination of sodium sulfate soundness loss and level of water absorption, four of the gravel aggregates classify as Type C and one as Type A.

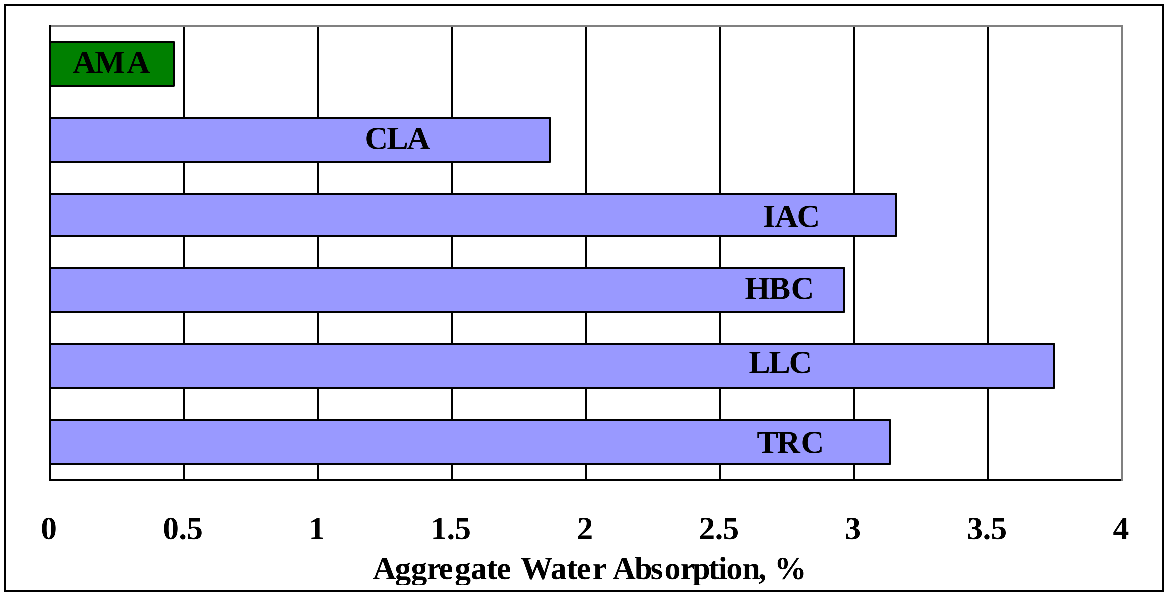

Figure 1 Percent Water Absorption for Different Aggregates

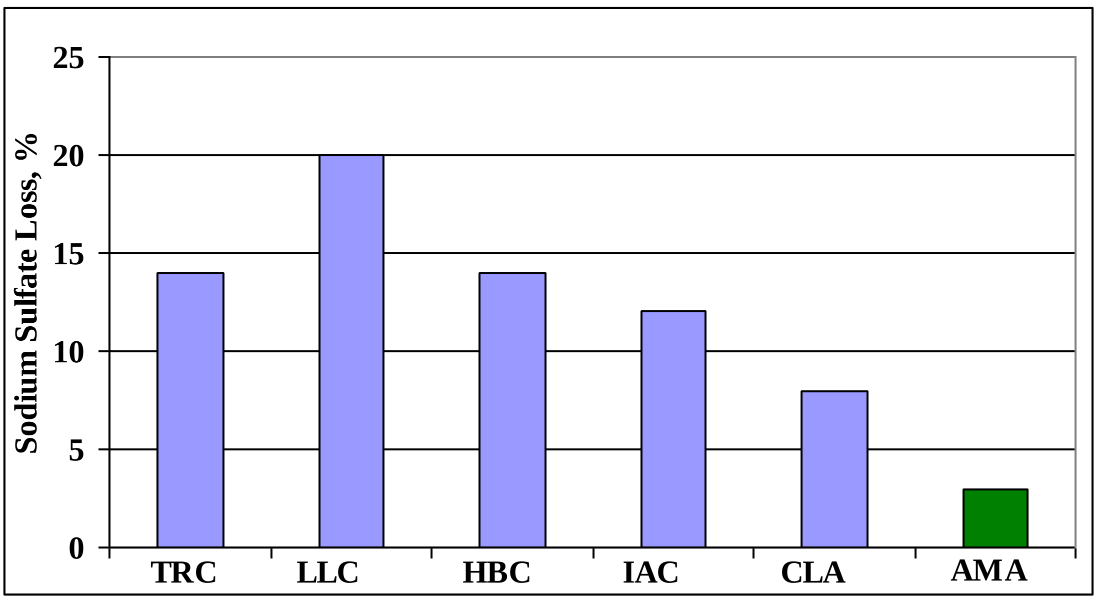

Figure 2 Percent Loss from Sodium Sulfate Soundness Test for Different Aggregates

Binder Source

The binder used in this research was a PG 64-22 from the United Refineries. This binder was used with all of the aggregates. The same binder modified with ¼ percent liquid antistripping agent (LAS) was also used in the study.

LABORATORY INVESTIGATION

Experiment Design

Through coordination with the PennDOT technical director, a series of tests were selected for evaluation of mixes produced under this research (Table 4).

Table 4 Testing Matrix to Evaluate Moisture Damage Resistance of Mixes

		Tests			
		1	2	3	
				Original	Revised
Agg. Source	Mixes	AASHTO T283	MMLS3	ECS/DM	RFT/DM
TRC	Control	√	√	√	√
	w/LAS	√			
	w/Lime	√			
	50/50	√			
LLC	Control	√			
	w/LAS	√			
	w/Lime	√			
	50/50	√			
HBC	Control	√	√		
	w/LAS	√			
	w/Lime	√			
	50/50	√			
IAC	Control	√			
	w/LAS	√			
	w/Lime	√			
	50/50	√			
CLA	Control	√	√	√	√
	w/LAS	√			
	w/Lime	√			
	50/50	√			

NOTES

PennDOT modified version of AASHTO T283 was used.
MMLS3: Model Mobile Load Simulator- 3rd Scale.
ECS/DM: Environmental Conditioning System combined with Dynamic Modulus.
RFT/DM: Repeated Freeze-Thaw Cycles combined with Dynamic Modulus Test.
LAS: Liquid Antistripping Agent.
50/50: A blend of #8 aggregates with 50 percent gravel and 50 percent limestone.

AASHTO T283 was selected because it is the most widely used test procedure for evaluation of mix resistance to moisture damage among different states, including PennDOT, and it is also a procedure for which PennDOT has a large amount of data available for various mixes. Of course, the procedure has been modified by PennDOT and this modified version, as given in PennDOT Bulletin 27, was used for this research.

The Model Mobile Load Simulator 3rd Scale was used to provide a means of accelerated testing of the mixes under both dry and wet conditions.

It was originally decided that the final laboratory test for the study should be dynamic modulus testing under the Environmental Conditioning System (ECS), as this was a system developed by Penn State under a research project sponsored by the National Cooperative Highway Research Program. Later, it was decided to conduct dynamic modulus tests under dry conditions and after conditioning with repeated freeze-thaw cycles, since the ECS system was not available at the time of conducting this research. In addition, repeated freeze-thaw conditioning is a less complicated procedure than the method used with ECS and might be more appropriate for mixes using absorptive aggregates, which is the case with this project.

Mix Design

The mix design to be used for this research project was selected by the PennDOT technical director of the project. This is a design that had been used for production and construction of hot-mix asphalt within District 1-0. The same gradation, as given in the mix design, was adopted for all aggregate sources. The blend required 54 percent of B-3 aggregate and 46 percent of AASHTO #8 aggregate. For the #8 aggregates, sizes from 9.5 mm to #16 were used and for the Type B-3 aggregate sizes ranged from #4 sieve to material passing #200 sieve. For all mixes, the same B-3 material was used (source: Troy Sand & Gravel). The course material (#8) was the portion that varied from source to source. Once gradation was established, design had to be verified for all mixes. For all mixes, the same binder, a PG 64-22 from United Refineries, was used. Replicates for each mix at trial asphalt contents were prepared and compacted at the design number of gyrations. It was decided to accept the binder content if the resulting air void would be within 3.5 to 4.5 percent. The mixing temperature was set at 155 ^{o}C, while the compaction temperature was set at 150 ^{o}C. The mixing took place in a 5-gallon bucket type mixer. The mix was then put into an oven to cure for 2 hours before compacting. After curing, the mix was then compacted under 75 gyrations using a Pine

Gyratory Compactor Model AFGC125X. A summary of mix design information is provided in Figure 3 and Table 5.

Mix Design Information

Sieves US Units	SI,mm Units	Combnd	SI^0.45
1.5	37.5	100.0	5.11
1	25	100.0	4.26
3/4	19	100.0	3.76
1/2	12.5	100.0	3.12
3/8	9.5	99.0	2.75
#4	4.75	63.0	2.02
#8	2.36	43.0	1.47
#16	1.18	29.0	1.08
#30	0.6	17.0	0.79
#50	0.3	8.0	0.58
#100	0.15	6.0	0.43
#200	0.075	4.0	0.31
pan	0	0.0	0.00

Aggregate Nominal Max Size	SP 9.5 mm
Aggregate Type	Gravel & Limestone
Binder Grade and Source	PG 64-22 United Refinery
Binder Content	Mix Dependent - See Table
Design Number of Gyrations	75

G_b (Binder Specific Gravity)@77 ^oF	1.0283
G_{sb} (Agg. Bulk Sp. Gr.)	Variable
G_{se} (Aggr. Effective Sp. Gr.)	Variable

Aggregate Size	Percent in Mix Design
#8	46
B3	54

Figure 3 Mix Design Information for All the Mixes included in the Study

Table 5 Design Verification Results

Aggregate Source	% Binder	% Voids at N_{des}
TRC-C	6.6	5.9
TRC-C	7.1	4.8
LLC-C	7.1	3.5
LLC-C	6.8	4.8
HBC-C	7.1	3.7
IAC-C	7.1	3.5
CLA-C	6.6	4.4
CLA-C	6.8	4.1

Aggregate Processing

With the start of the project came the aggregate processing stage. The PennDOT technical director was directly involved with expediting aggregate shipment to the LTI testing facilities at Penn State. The buckets of aggregate received at the laboratory were inventoried. A table was made to keep track of the progress of the aggregate processing stage. There were a total of eighty-eight 5-gallon buckets of aggregate, and one 5-gallon bucket of lime. Throughout the sieving process, it was noticed that the amount of fine material from B3 and that retained on #16 from #8 aggregate was in short supply for one of the sources. Afterwards the researchers received seven additional 5-gallon buckets of that aggregate.

During the first part of processing, the aggregates were oven dried, followed by sieving. The aggregates were dried in ovens overnight at 110 ^{o}C at a rate of several buckets at a time. After drying and cooling, the material was then sieved using either the MaryAnn sifter or Gilson Test Master shaker, using sieve sizes from 12.5 mm to #200. Following sieving was the washing stage, as the fine dust was washed off the aggregates in the size range down to and including #30. The aggregates were washed over a stack of 3 sieve sizes #8, #30 and #200. This step was done using water and a deflocculating agent, pouring the liquid through #30 and #200 sieves. The remaining materials on the screens were separated.

After the aggregate was washed, it was then dried in ovens again overnight at 110 ^{o}C, several buckets at a time. Next, the aggregate was put into storage bins for use during the project. Any material that would not fit into the storage bins was put back into 5-gallon buckets. All of the aggregate bins and buckets were labeled according to the source from which the material came.

The specific gravities of the fine and coarse aggregates were measured according to AASHTO T84 and T85, respectively, as reported in Table 6.

Table 6 Measured Specific Gravities of Aggregates

Aggregate Source	G_{sb}	G_{sa}	Absorption %
Troy Sand Gravel, #8	2.469 2.471	2.672 2.682	3.1 3.2
Average	**2.470**	**2.677**	**3.1**
Conneaut Lake Gravel, #8	2.574 2.560	2.703 2.689	1.9 1.9
Average	**2.567**	**2.696**	**1.9**
Lakeland Gravel, #8	2.429 2.417	2.672 2.657	3.8 3.7
Average	**2.423**	**2.665**	**3.7**
Hasbrouck Gravel, #8	2.499 2.493	2.702 2.689	3.0 2.9
Average	**2.496**	**2.695**	**3.0**
IA Construction Gravel, #8	2.480 2.472	2.690 2.682	3.2 3.2
Average	**2.476**	**2.686**	**3.2**
Allegheny Mineral Limestone, #8	2.710 2.704	2.743 2.740	0.4 0.5
Average	**2.707**	**2.741**	**0.5**
Troy Sand Gravel, B3	2.543 2.498	2.679 2.657	2.0 2.4
Average	**2.521**	**2.668**	**2.2**

Lime Treatment

It was mentioned that for each of the aggregate sources, four levels of treatment were used:

- control mix with no treatment,
- Binder modified with one-quarter percent liquid anti-stripping agent,
- Replacement of 50 percent of +#8 gravel with limestone, and
- treatment of the aggregate hydrated lime.

In the case of lime treatment, one percent lime by weight of the aggregate replaced one percent of material passing #200 sieve. The following procedure was followed to incorporate lime into the mix.

1. Determine percent water absorption of the aggregate blend.
2. Moisten the aggregate blend with 2.5% water above the percent water absorption. For example, if aggregate blend has 3 percent water absorption, add 5.5 percent water. Mix thoroughly to ensure uniform dampness of the aggregate blend.
3. Add lime to the damp aggregate and mix thoroughly. Lime should be added as one percent of the dry weight of the aggregate blend.
4. Cure for 24 hours at ambient temperature.
5. Heat the aggregate blend as is typically done for any aggregate blend and process for mixing with binder.

Testing Program

Table 7 provides a summary of test procedures used for this project.

Table 7 Tests Conducted on the Cores for WO-11 Project

Designation	Standard Method of Test for
AASHTO T 283-07 PTM Version	Resistance of Compacted Asphalt Mixtures to Moisture-Induced Damage
AASHTO T 166-05	Bulk Specific Gravity of Compacted Hot-Mix Asphalt Using Saturated Surface-Dry Specimens
AASHTO T 209-05	Theoretical Maximum Specific Gravity and Density of Hot-Mix Asphalt Paving Mixtures
AASHTO T 164-05	Quantitative Extraction of Asphalt Binder from Hot-Mix Asphalt (HMA
AASHTO TP 69-04	Standard Method of Test for Bulk Specific Gravity and Density of Compacted Asphalt Mixtures Using Automatic Vacuum Sealing Method
AASHTO T 30-05	Mechanical Analysis of Extracted Aggregates
AASHTO T 84-08	Standard Method of Test for Specific Gravity and Absorption of Fine Aggregate
AASHTO T 85-08	Standard Method of Test for Specific Gravity and Absorption of Coarse Aggregate

Indirect Tensile Testing

All five mixes (TRC, LLC, HBC, IAC, and CLA) underwent the indirect tensile testing according to the standard of AASHTO T283. Each mix contained four groups with different additives (Control, with LAS, with lime, and 50/50), and each of the four groups contained six Superpave Gyratory Compacted Cylindrical (SGC) specimens.

The six specimens of each set were divided into two subsets. One subset was tested in dry conditions for indirect-tensile strength. The other subset was subjected to vacuum saturation and a freeze cycle, followed by a hot-water soaking cycle, before being tested for indirect-tensile strength. The ratio of retained tensile strengths of conditioned and dry subset was calculated, as shown in equation 1, to determine whether the tested mix was resistant to moisture damage or not. The criterion was that when TSR is greater than 0.80, the mix can be viewed as resistant to moisture damage; if TSR is less than 0.80, the mix can be viewed as susceptible to moisture damage.

$$TSR = \frac{Average\ Tensile\ StrengthofUnconditionedSpecimens}{Average\ Tensile\ StrengthofConditionedSpecimens} \qquad \textbf{(1)}$$

<u>Testing Equipment</u>

Test equipment included devices for mixing and compacting SGC specimens; vacuum containers with a vacuum pump, balance, and water tank for AASHTO T166; water bath with constant temperature of 60 ^{o}C; freezer; plastic film for wrapping specimens; leak-proof plastic bags to enclose saturated specimens; INSTRON series 5583 loading frame, which is the loading jack and ring dynamometer to provide a range of accurately controllable rates of vertical deformation; and steel loading strips used to hold the specimen when performing the indirect tensile test (Figures 4 and 5).

Figure 4 SGC Specimen Set in the Loading Frame for Indirect Tensile Testing

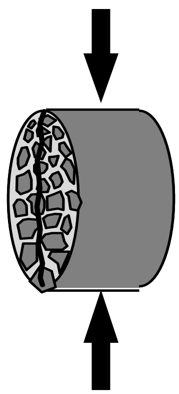

Figure 5 Testing in Diametral Direction to Determine Indirect Tensile Strength

Specimen Preparation

a. Prepare enough batches for six specimens each set.

b. Heat batches and binder to mixing temperature (155 ^oC) and start mixing.

c. After mixing, the mixture should be placed at room temperature for 2±0.5 hours. Then place the mixture in a 60±3 ^oC oven for 16±1 hours for curing.

d. After curing, place the mixture in an oven for 2 hours±10minutes at the compaction temperature (148 ^{o}C) prior to compaction. Compact the specimens to the height of 95-mm and store the specimens for 24±3 hours.
e. Divide 6 specimens into two subsets. One subset will be tested dry and the other will be partially vacuum-saturated, subjected to freezing and soaked in hot water before testing.

Specimen Conditioning

f. The conditioned specimens should be placed in the vacuum containers with spacer separately on the bottom and at least 25 mm of water above the surface of the specimens. Apply a vacuum of approximate 22 in-Hg partial pressure for 30 minutes. Remove the vacuum and leave the specimen submerged in water for 5 to 10 minutes.
g. Determine the mass of saturated, surface-dry specimen after partial vacuum saturation using the AASHTO T166 method.
h. Calculate the volume of absorbed water in cubic centimeters.
i. Determine the degree of saturation.
j. Cover each of the conditioned specimens with plastic film and place them in a plastic bag containing 10±0.5 mL of water and seal the bag.
k. Place the bag in a freezer at -18±3 ^{o}C for minimum 16 hours.
l. Place the specimens in a 60±1 ^{o}C water bath for 24±1 hours and remove the plastic bag and film as soon as possible.
m. Place specimens in a water bath at 25±0.5 ^{o}C for 2 hours±10minutes.
n. The dry specimens would be stored at room temperature. They are wrapped in a heavy-duty, leak-proof plastic bag and put in a 25±0.5 ^{o}C water bath along with the conditioned specimens for 2 hours±10 minutes with a minimum 25-mm of water above their surface before being tested with the indirect tensile tester.

Testing Protocol

a. Remove the specimen from the water bath. Place it between the steel strips. Apply load to the specimen by means of a constant rate of movement of the testing machine head at 50-mm per minute.
b. Record the maximum compressive strength noted on the testing machine and continue loading until a vertical crack appears. Remove the specimen from the machine and pull it apart at the crack. Inspect the interior surface for evidence of cracked or broken aggregate.
c. Calculate the ratio of average tensile strength of the dry and conditioned subsets, known as the tensile strength ratio (TSR).

Model Mobile Load Simulator, 3rd Scale

As shown in the beginning of this chapter, three kinds of aggregates (TRC, HBC, and CLA) were chosen to be tested using the MMLS3 equipment.

The MMLS3, manufactured by MLS test systems in South Africa, is one of the small-scale accelerated trafficking devices. It contains four pneumatic tires that can be inflated to a maximum pressure of 700 kPa (approximately 100 psi). For this project, the tires were inflated to between 600 and 650 kPa (approximately 87 to 94 psi). The tires move on bogies to apply unidirectional moving wheel loadings to the briquettes (trimmed SGC specimens). The suspensions of these wheels are calibrated so that during trafficking, they apply a load of 2.7 kN on the specimens.

To run the test, SGC specimens have to be trimmed and put in the test bed, which allows a total of nine trimmed specimens to form one testing path for testing at the time same (Figure 6). However, to prevent the approaching and departing impacts from the wheels, specimens placed on both sides are replaced by dummy specimens. A thermocouple is inserted in one dummy specimen to monitor the pavement temperature. Therefore, in this project, the test bed is capable of testing a total of seven briquettes at the same time. The seven briquettes consist of three specimens of TRC mix, three specimens of CLA mix, and one specimen of HBC mix.

The MMLS3 can run the accelerated trafficking test under two different environmental conditions, wet and dry. In this project, both wet and dry conditions were tested at a temperature of 52 °C up to at least 400,000 trafficking cycles in an attempt to observe and compare the effect of these different conditions by measuring the rutting of the briquettes. Figure 7 demonstrates how the MMLS3 is positioned at the top of the test bed.

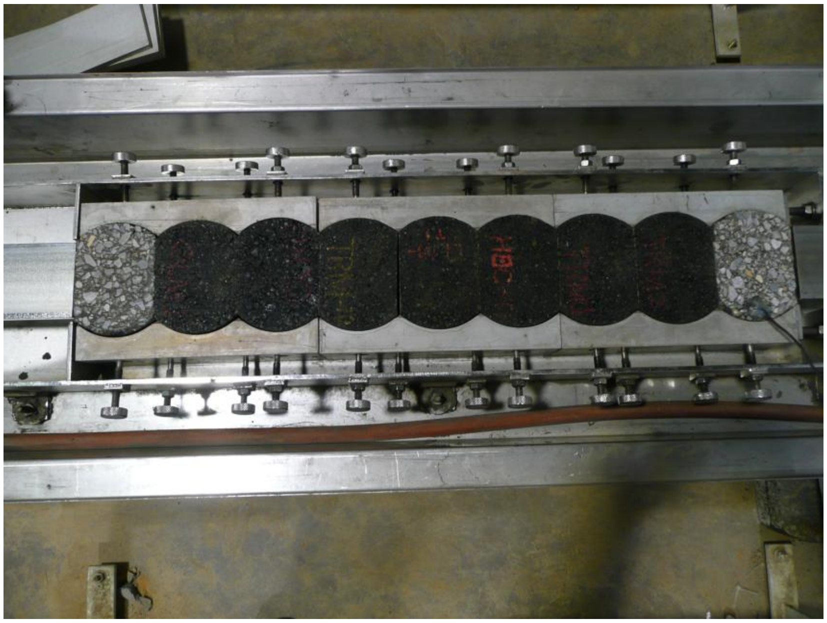

Figure 6 Briquettes are Placed in the Test Bed before Mounting MMLS3

Figure 7 MMLS3 on Top of the Test Bed

Testing Equipment

Testing equipment included:

a. MMLS3;
b. Test bed for placing briquettes, and for water bath; it also serves as the base for the MMLS3 when testing;
c. Wet heater system;
d. Dry heating/cooling system;
e. Thermocouples;
f. Environmental chamber; and
g. Profilometer for measuring the rutting of testing briquettes.

Specimen Preparation

Specimens used for this test were mixed and compacted under the same method used to prepare specimens for the control group in the indirect tensile test. However, the specimens were compacted to 75-mm high and were trimmed by saw in order to fit in the briquette spaces of the test bed. The briquettes obtained from SGC specimens were 105-mm wide and 150-mm long, as shown in Figure 8.

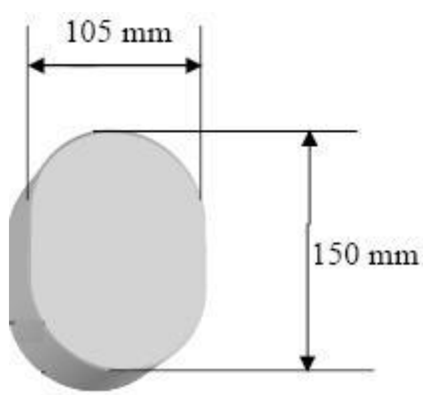

Figure 8 Typical Dimension of the MMLS3 Briquette

Testing Protocol

The testing procedure is explained in two parts, wet setup and dry setup. Briquette configurations, trafficking loads, and testing of trafficking cycles for both setups remained the same.

For wet setup (Figure 9), the wet heater system was used; hot water flowed into the test to submerge and heat the briquettes to the target temperature (around 52 ^{o}C to 53 ^{o}C). Excessive water was pumped back to the heater to be reheated. The environmental chamber was incorporated to cover the whole testing system for better insulation. When the temperature reached the target value and remained unaltered, the test could be started by setting up the trafficking cycles on the control panel. Measurement of rutting using the profilometer should be done between each trafficking cycle in order to record the rutting-development process.

Figure 9 Wet Setup Covered by Environmental Chamber

For dry setup (Figure 10), a dry heating system was adopted. The dry heating system blew out hot air at the input temperature to the briquettes. The environmental chamber was incorporated here also for insulation purposes. Likewise, when the temperature of the briquettes reached the target value and remained unchanged, the test could be started. Measurement of rutting using the profilometer was done between each trafficking cycle in order to record the rutting development process.

Figure 10 Dry Setup for MMLS3 Testing

Dynamic Modulus Testing with Repeated Freeze-Thaw Cycles

Two of the control mixes were considered for this testing, one of the mixes (TRC) with a high level of aggregate water absorption and sodium sulfate soundness loss, the other with a low level for these two parameters (CLA). The specimens were prepared and tested, first under dry conditions. The specimens were subsequently exposed to one freeze-thaw cycle. The procedure outlined under AASHTO T283 with PennDOT modification was followed to condition specimens. These conditioned specimens were tested for dynamic modulus and then were subject to a second round of freeze-thaw conditioning. The specimens were tested again for dynamic modulus. This sequence of testing and conditioning was repeated three times.

This section provides an explanation of how the specimens for this testing were prepared and how the dynamic modulus tests were conducted. Results are discussed in Chapter 3.

Specimen Preparation

The dynamic modulus test specimens were manufactured by coring and sawing 100-mm-diameter (3.94-inch) by 150-mm-high (5.90-inch) test specimens from the middle of gyratory compacted specimens that were 150-mm (5.90-inches) in diameter by 165-mm (6.5-inches) high

(Figure 11). The procedure for preparing dynamic modulus specimens is described in AASHTO TP62.

Figure 11 Dynamic Modulus Specimens are Cored and Sawed from SGC Specimen

Dynamic Modulus Test

The dynamic modulus testing was conducted with a uniaxial sinusoidal load inducing approximately 100 microstrain in the specimen (Figure 12). All dynamic modulus tests were conducted at 25 ^{o}C. Selection of the 25 ^{o}C test temperature was based on findings from the research under NCHRP 9-29, which concluded that dynamic modulus testing at moderate temperatures close to 25 ^{o}C produced less variability in results compared with tests at extreme temperatures such as -10 C or 40 ^{o}C, respectively. Specimen setup and temperature control is also more easily managed at moderate temperatures. The loading frequencies for each specimen were 10, 5, 2, and 1 Hz applied in decreasing order.

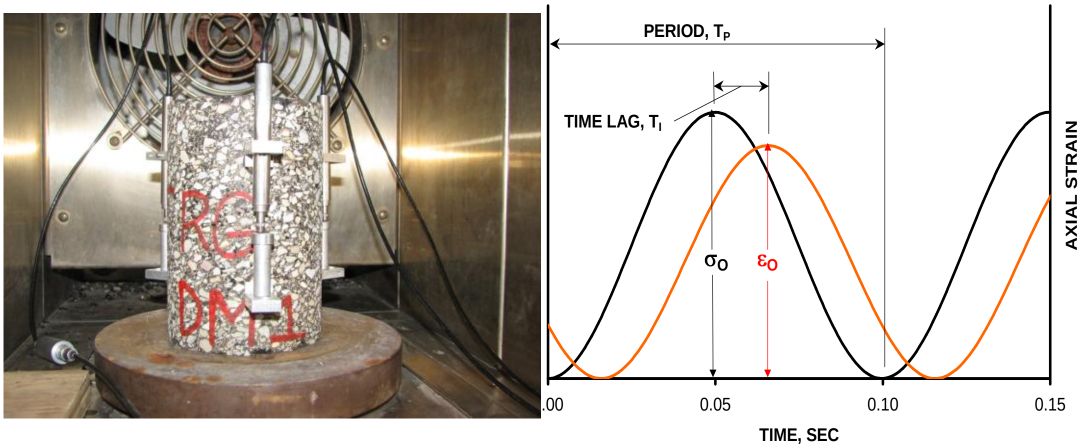

Figure 12 Specimen Setup for Testing and the Corresponding Sinusoidal Load

The dynamic modulus and phase angle are defined by Equations 2 and 3, respectively.

$$|E^*| = \frac{\sigma_0}{\varepsilon_0} \qquad (2)$$

$$\phi = \left(\frac{T_i}{T_p}\right) \times 360 \qquad (3)$$

where:

$|E^*|$ = dynamic modulus

σ_0 = amplitude of applied sinusoidal loading

ε_0 = amplitude of resulting sinusoidal strain

ϕ = phase angle, in degrees

T_l = time lag, in seconds

T_p = period of sinusoidal loading, in seconds

Three linear variable displacement transducers (LVDTs) were used at 120^o to capture deformation of the specimen during both dynamic modulus testing and repeated loading of the conditioning phase (Figure 13). Dynamic modulus testing parameters are presented in Table 8.

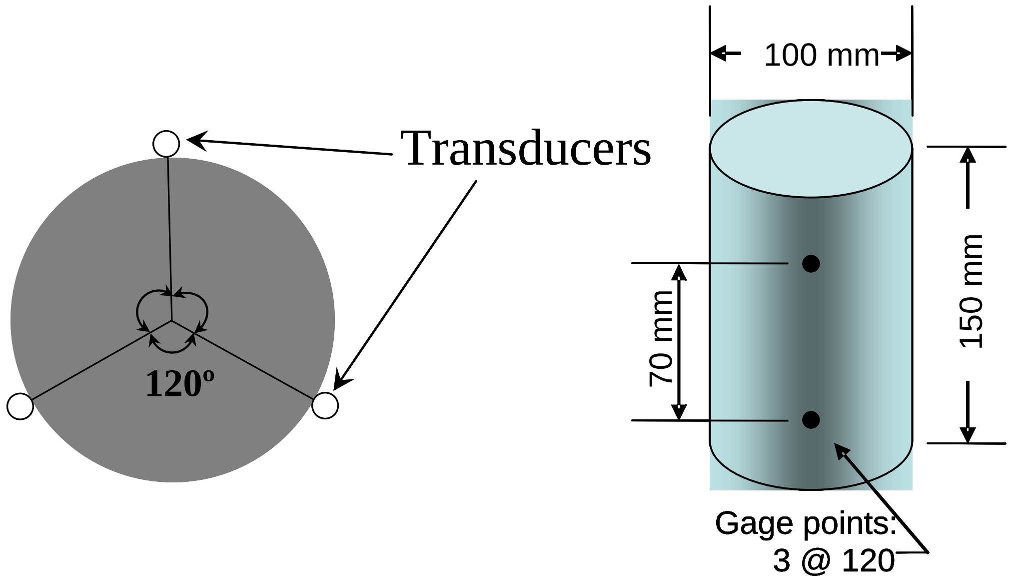

Figure 13 Schematics Showing Configuration of LVDTs on the Specimen

Table 8 Description of Parameters for WO-11 Dynamic Modulus Testing

Parameter	Value/Type
Temperature	25 °C
Load Pattern	Sinusoidal
Frequencies	10, 5, 2, and 1 Hz
Load Level	Variable
Displacement Measurement	3 LVDTs @ 120 ° Axial Direction
Measurement Span in Axial Direction	70 mm
Strain Level	50 ± 20 microstrain

CHAPTER 3
ANALYSIS, INTERPRETATION, AND FINDINGS

This chapter covers the results of laboratory investigation as part of this research project.

EFFECT OF CURING TIME ON MAXIMUM THEORETICAL SPECIFIC GRAVITY

According to PennDOT Bulletin 27, curing time for determination of maximum theoretical specific gravity (Gmm) should be extended beyond the traditional 2-hour oven curing time if water absorption of the coarse aggregates used in the design exceeds 1.5 percent as determined by AASHTO T 85. If absorption exceeds this limit, mixes must be cured for 6 hours unless it can be demonstrated that a lower oven curing time yields maximum specific gravity values that differ from the 6-hour cure time by less than 0.010. Table 9 indicates that for three of the aggregates used in this study, the Gmm values for 6-hour curing differ more than 0.01 from the Gmm values for 2-hour curing. However, in all cases the difference is less than 0.015.

Table 9 G_{mm} Values with 2-hour and 6-hour Curing for the 5 Mixes

	TRC	LLC	HBC	IAC	CLA
Asphalt Content %	7.1	6.9	6.7	6.9	6.9
Measured G_{mm} with 2-hour curing	2.361	2.332	2.354	2.347	2.378
Measured G_{mm} with 6-hour curing	2.355	2.346	2.368	2.359	2.385

RESULTS OF TENSILE STRENGTH TESTS

The results of the PennDOT-modified version of the AASHTO T283 test on the mixtures from the five aggregate sources with the PG 64-22 binder are summarized in Appendix A, where the data obtained from individual specimens are presented along with averages and standard deviations for air voids, saturation, and tensile strength. It can be seen from those results that one of the specimens of the control mix for TRC yields a considerably high value compared to the other two, obviously being an outlier. This value was excluded from all the analysis discussed below.

The T283 test results were first checked against the acceptable levels of variability reported in PennDOT Bulletin 27. According to the PennDOT version of the test, the coefficient of variability for strength of dry and conditioned specimens should not exceed 12 and 24 percent, respectively. Results presented in Appendix A indicate that for all conditioned specimens, the criterion was met.

For unconditioned (dry) specimens, out of the 15 mixes tested, 4 exceeded the 12 percent limit (12.3, 15.7, 15.0, and 12.5). Due to time and budget restrictions these values were considered acceptable for further investigation and no new mixes were generated. Unfortunately, no precision statement currently exists for the AASHTO T283 procedure. A precision statement is published in ASTM D4867, based on which the within-laboratory standard deviation for tensile strengths on dry and conditioned specimens is 55 kPa. However, this value was developed for 100-mm-diameter specimens and could not be applied to T283 specimens, which had a diameter of 150 mm. Therefore, no appropriate statistical test could be utilized to check the standard deviation of the measured tensile strengths.

Tensile strength ratios are presented in Figure 14. Several important observations can be made based on the results shown in this figure. First, it can be observed that tensile strength ratios for three of these gravel sources are less than 0.80, indicating susceptibility of these mixes to moisture damage. The control mix of HBC produces a TSR value of 0.87. For the control mix of IAC, the TSR is obtained at 0.51 percent level, a considerably low value. The second important observation is that the liquid antistripping agent is making a significant improvement in TSR value and, therefore, in resistance to moisture damage. The slightest improvement (10% increase in TSR) due to the usage of LAS is for the HBC aggregate and the largest improvement (56% increase in TSR) is for the CLA aggregate. It should be noted that the CLA aggregate is the only gravel aggregate out of this group which is classified as a Type A aggregate. In summary, for all aggregates TSR is increased when LAS is used, and for two of the failing sources, LAS increases TSR to a level exceeding 0.8, making the mix an acceptable one. The only mix that still remains in the failing range after the addition of LAS is the IAC source, even though TSR is increased as a result of adding LAS. The third important observation is that using a 50/50 blend of gravel and limestone for the #8 portion of the aggregate delivered mixed results in terms of its impact on TSR. Use of such a blend yielded increased TSR values (improvement in moisture damage resistance) for TRC, LLC, and CLA aggregates, even though the level of improvement observed using a 50/50 blend was lower than that observed using LAS. For the HBC and IAC mixes, the 50/50 blend decreased the TSR values compared with the control mix. It is also observed that lime treatment caused improvement in moisture damage resistance only with the Conneaut Lake material. In all other cases, a reduction in TSR is observed as a result of adding lime to the aggregate. Hydrated lime in general has been shown by other research efforts to improve moisture damage resistance of hot-mix asphalt concrete. The reason such behavior was not observed for the mixes used in this study is unknown at this time. The poor performance of lime-treated mixes of this study could have come from the impact of the interaction between the fine material in the mix and the lime, as the same fine material was used for all mixes. However, further investigation is needed to truly identify why adding lime did not improve moisture damage resistance.

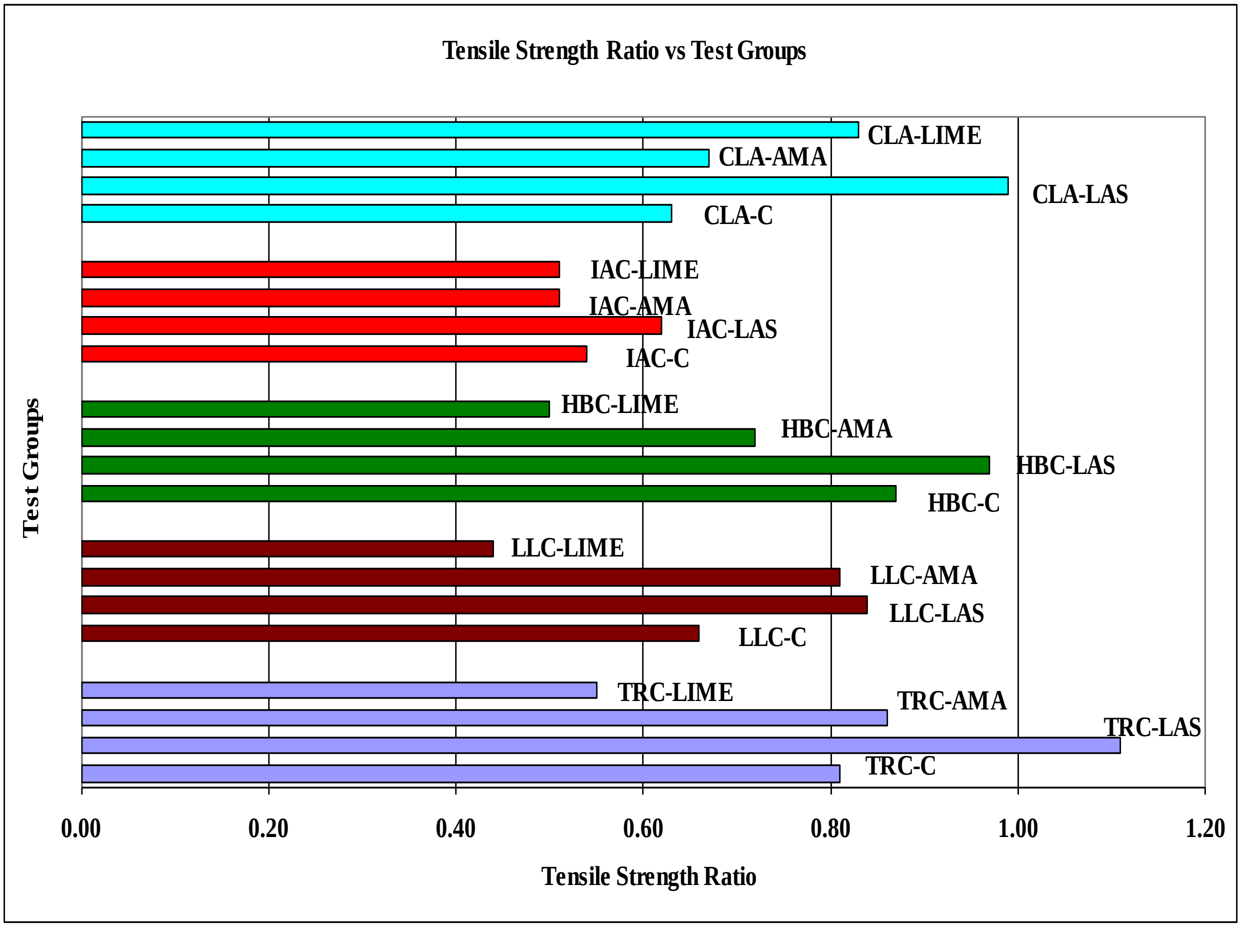

Figure 14 Tensile Strength Ratio from AASHTO T283 Tests for Different Mixes

Statistical Comparison of Mean Tensile Strength of Dry versus Conditioned Specimens

It is reasonably assumed that the tensile strengths follow a normal distribution. A statistical student t-test was conducted to determine whether the tensile strength of the conditioned specimens was different from that of the dry (unconditioned) specimens. The null and alternative hypotheses to be tested were as follows:

$$H_0: \quad \mu_{dry} = \mu_{conditioned}$$
$$H_1: \quad \mu_{dry} > \mu_{conditioned}$$

where μ_{dry} and $\mu_{conditioned}$ refer to the true mean of the tensile strength for dry and conditioned specimens, respectively. From experience, it is known that conditioned samples of the T283 procedure carry a different variability compared with unconditioned specimens. As a result, the t-test applied here is the one for two groups with unequal variances. The t statistic is calculated as

$$T = \frac{\overline{X} - \overline{Y}}{\sqrt{\frac{S_X^2}{n_1} + \frac{S_Y^2}{n_2}}} \tag{4}$$

where $\overline{X}$ and $\overline{Y}$ are the average tensile strengths for dry and conditioned specimens of each mix, respectively. S_x^2 and S_y^2 are the respective variances. The number of samples per group is shown by n_1 and n_2 ($n_1 = n_2 = 3$ for this experiment). Table 10 shows the results. It can be seen that in all cases for the control mixes, except for the HBC aggregate, statistically, there is a significant difference between the dry and wet strengths, emphasizing the previous observation of low TSR values. The control mix of HBC yields a TSR of 0.87, indicating proper moisture damage resistance, and the reason why no significant difference is observed for the control mix of HBC, as reported in Table 10. It can also be observed that for mixes with LAS, there is no statistically significant difference between the dry and wet strengths, emphasizing improvement gained through the use of LAS. The exception is the IAC aggregate, for which significant difference is observed between dry and wet strengths, indicating that LAS has not resulted in any significant improvement for this mix. For lime-treated mixes, except for the Conneaut Lake mix, there is statistically a significant difference between the dry and wet strength, implying significant reduction in strength of the mix due to water conditioning.

Table 10 Hypothesis Test Results for Difference Between Conditioned and Unconditioned Specimens

Mixture	Average Strength of Dry Specimens, KPa	Average Strength of Conditioned Specimens, KPa	t-Statistic	Critical t-Statistic at α =0.05 level		Conclusion: Is Conditioned Strength Significantly Different from Unconditioned Strength?
				One-tailed	Two-tailed	
TRC-C	1773	930	3.46	2.920	4.30	Yes
TRC-LAS	824	919	-1.109	2.920	4.30	NO
TRC-AMA	682	587	1.688	2.353	3.18	NO
TRC-LIME	928	507	7.50	2.132	2.776	YES
LLC-C	716	470	5.098	2.353	3.18	YES
LLC-LAS	780	655	1.366	2.132	2.78	NO
LLC-AMA	588	477	2.904	2.132	2.78	YES
LLC-LIME	931	413	9.480	2.920	4.302	YES
HBC-C	855	747	1.163	2.353	3.18	NO
HBC-LAS	759	733	0.342	2.353	3.18	NO
HBC-AMA	614	440	3.309	2.132	2.78	YES
HBC-LIME	862	435	7.945	2.353	3.182	YES
IAC-C	886	440	10.736	2.353	3.18	YES
IAC-LAS	714	442	6.697	2.920	4.30	YES
IAC-AMA	685	352	11.995	2.353	3.18	YES
IAC-LIME	802	408	7.945	2.353	3.382	YES
CLA-C	859	543	3.636	2.353	3.18	YES
CLA-LAS	1292	1277	0.117	2.353	3.18	NO
CLA-AMA	702	474	3.666	2.132	2.78	YES
CLA-LIME	945	786	2.564	2.920	4.302	NO

Impact of Saturation Level

It was explained in Chapter 2 that short-term water conditioning of the specimens was accomplished through application of a vacuum to the specimen for 30 minutes at 25 ºC. There are two options available to induce partial saturation to the specimen. One is through controlling the degree of saturation under vacuum, as is used in AASHTO T283 or ASTM D4867. The second approach is through controlling the time duration of applying the vacuum. This second approach was used in this research as is the practice followed by PennDOT Bulletin 27.

It can be observed that the degree of saturation varies among mixes and within the replicate specimens of the same mix (Table 11). Statistical one-way analysis of variance indicates that not all mixes have similar saturation levels (Table 12). The results, summarized in Table 12, show a greater between-mixture variation compared to within-mixture variation, suggesting the saturation level is not the same for the mixes used in this study. It should be noted that the saturation level of the control mix of TRC is obviously lower than all the others and was not included in the one-way analysis of variance results reported in Table 12. This low level of saturation was the result of using a low level of partial vacuum applied at the time of the vacuum saturation process.

TABLE 11 Degree of Saturation (%) for Different Mixes

Mix	**Sat Level %**	**Mix**	**Sat Level %**	**Mix**	**Sat Level %**	**Mix**	**Sat Level %**	**Mix**	**Sat Level %**
TRC Control	41.1 31.3 30.3	CLA Control	67.6 73.0 73.3	HBC Control	81.9 82.9 79.5	LLC Control	92.5 90.2 85.8	IAC Control	71.4 71.6 74.0
TRC with LAS	72.5 71.9 72.3	CLA with LAS	65.3 67.8 72.8	HBC with LAS	74.3 75.1 78.5	LLC with LAS	80.8 78.4 79.6	IAC with LAS	74.5 72.9 74.1
TRC with AMA	83.5 86.2 85.7	CLA with AMA	92.0 92.2 92.4	HBC with AMA	90.7 91.2 95.8	LLC with AMA	79.2 81.6 77.2	IAC with AMA	75.6 75.1 77.6
TRC with LIME	77.9 77.6 75.8	CLA with LIME	74.0 75.2 73.8	HBC with LIME	71.1 75.9 72.0	LLC with LIME	75.3 72.8 78.6	IAC with LIME	75.0 78.7 77.6

TABLE 12. One-Way Analysis of Variance for Degree of Saturation for TSR Specimens

Source of Variation	SS	df	MS	F	P-value	F crit
Between Groups	2433.245	18	187.1727	32.27	1.3E-17	1.88
Within Groups	131.5344	38	4.697658			
Total	2564.779	56				

Figure 15 indicates the saturation level for different mixes. The error bars shown are 95 percent confidence intervals for each mixture. It is reasonably assumed that mixes with overlapping error bars have similar saturation levels. With this logic, three groups of saturation are observed. One is the TRC-C mix, which is obviously at a very low saturation level because of applying a low partial vacuum level. The other group at the high end includes TRC-AMA, HBC-AMA, and CLA-AMA mixes. Others fall in the last group. As shown in Figure 16, there is no evidence that differences observed in degree of saturation are the result of differences observed in the air void levels among the various mixes.

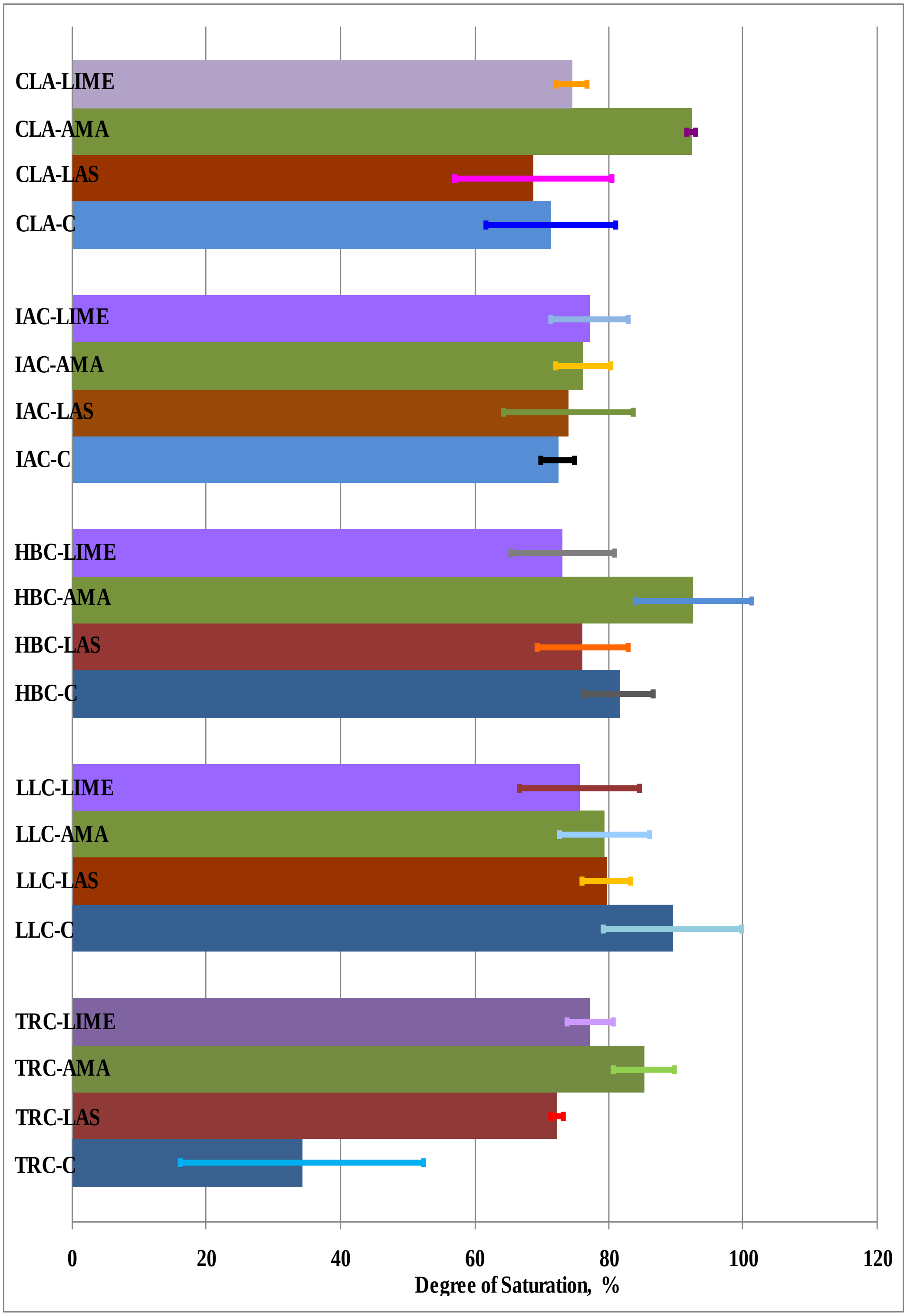

Figure 15 Relationship between Air Voids and Saturation Levels

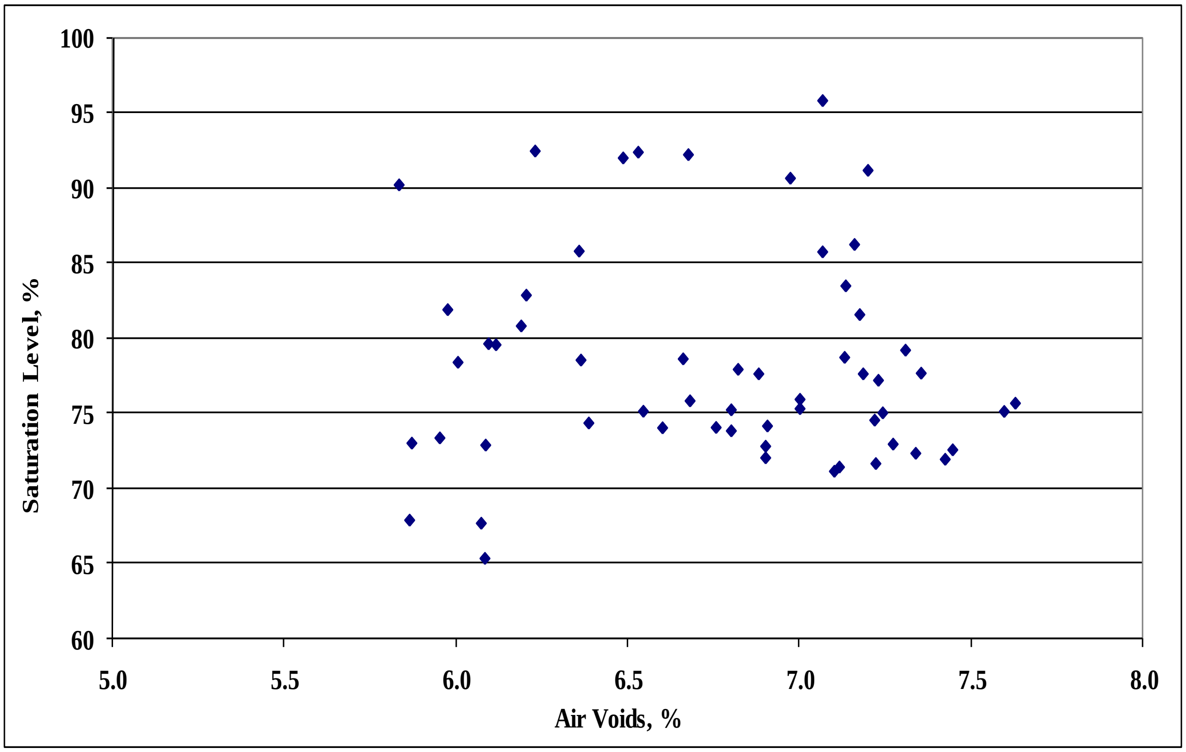

Figure 16 Relationship between Air Voids and Saturation Levels of All Conditioned Specimens

Scatter plots in Figure 17 exhibit the relationship between the saturation level and the measured tensile strength ratio. It can be seen that no particular trend could be established for such a relationship, implying that differences observed in the tensile strength ratios of various mixes are not the result of differences observed in saturation levels.

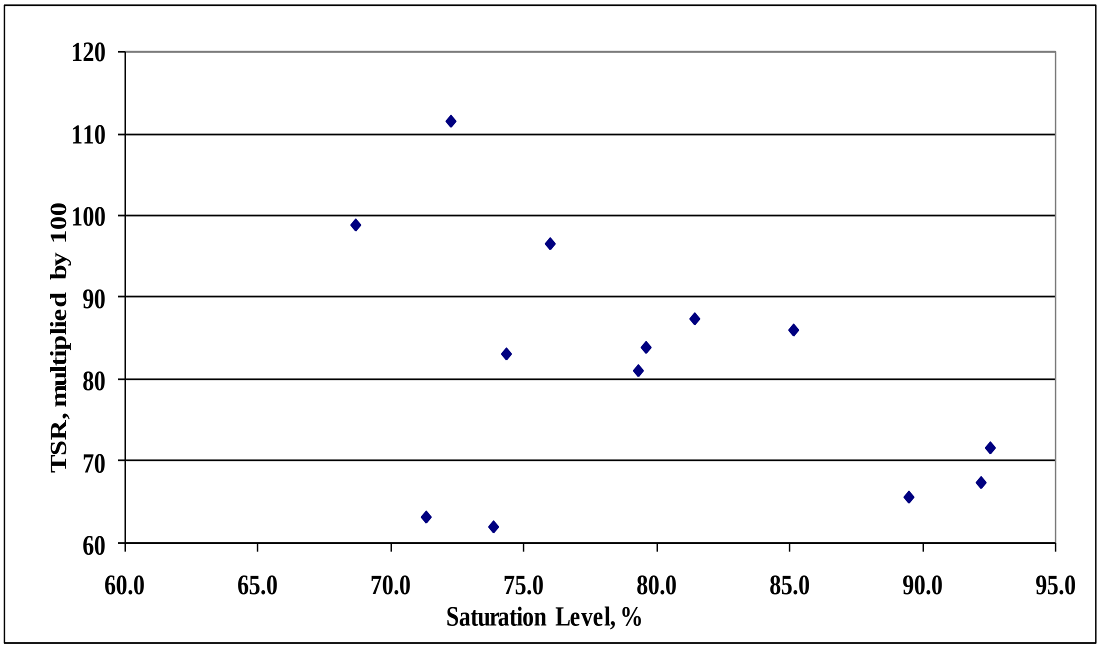

Figure 17 Relationship between the Saturation Level and the Measured Tensile Strength Ratio

Air Void Analysis

Additional analyses of the data were conducted to determine if there was bias in the testing that may have influenced the results. The first analysis that was performed was a two-way analysis of variance to see if the air void contents were significantly affected by mixture type or conditioning. The results, summarized in Table 13, show that there is no significant difference in air voids between dry and conditioned specimens (conditioning) but there is significant difference in air voids among various mixes at a 5 percent significance level. The fact that dry and conditioned mixes have similar air voids is shown graphically in Figure 18, which presents 95 percent confidence levels on the mean air void content of the mixtures for unconditioned and conditioned specimens. The confidence intervals overlap, implying that the air void contents were similar for the various mixtures and for dry and conditioned specimens.

Table 13 Two-Way Analysis of Variance for Air Voids

ANOVA Source	SS	df	MS	F	P-value	F crit
Mixture	21.77786	14	1.555562	43.01698	5.82E-26	1.860242
Conditioning	0.145571	1	0.145571	4.025577	0.049329	4.001191
Interaction	0.72832	14	0.052023	1.438622	0.163959	1.860242
Within	2.169694	60	0.036162			
Total	24.82145	89				

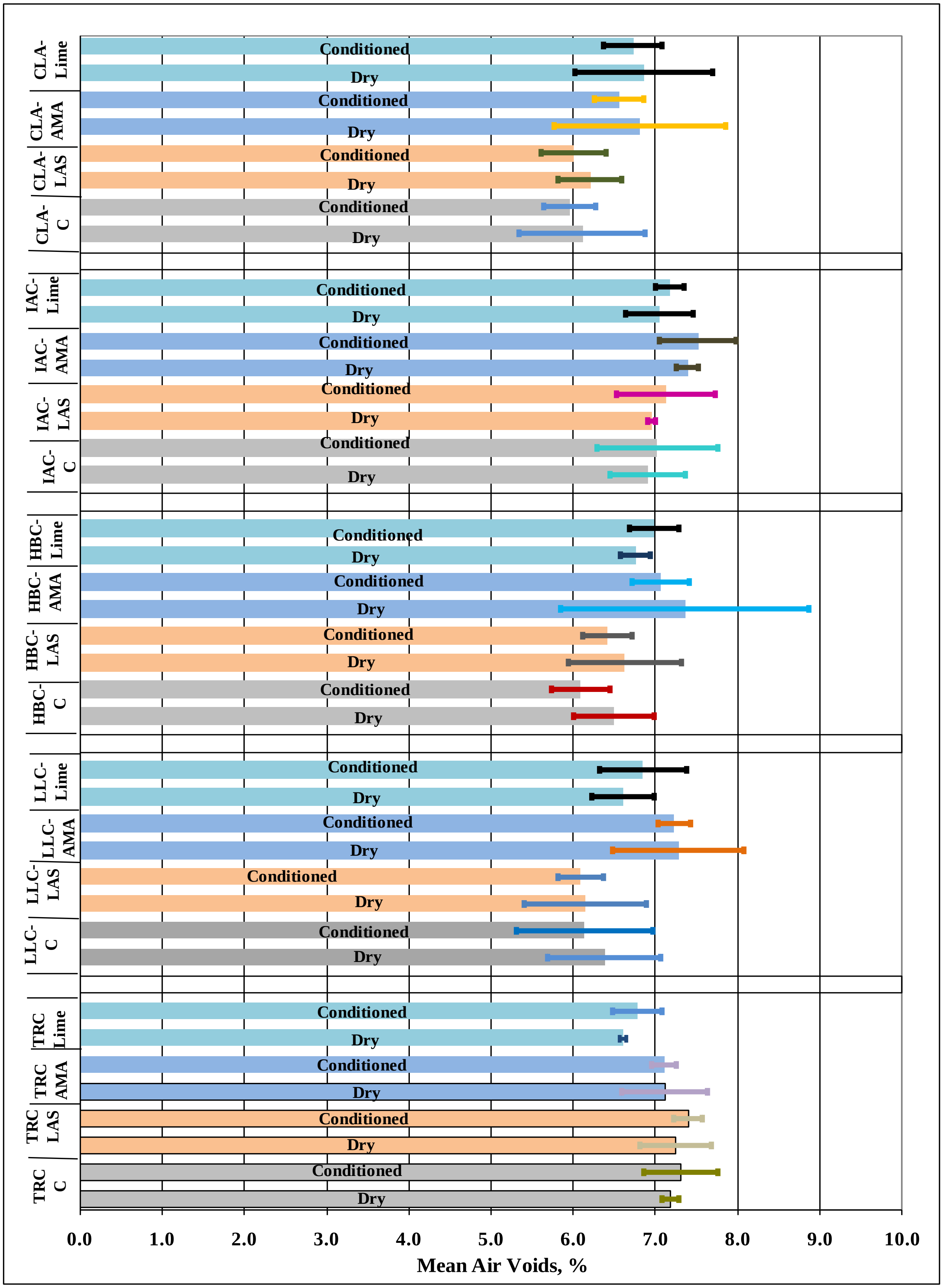

Figure 18 Ninety-five Percent Confidence Interval for Mean Air Voids

Even though statistically there is a significant difference in the air voids of the various mixes, this difference has not affected the TSR results, and as shown in Figure 19 no correlation was observed between the air voids of the conditioned mixes and the resulting TSR.

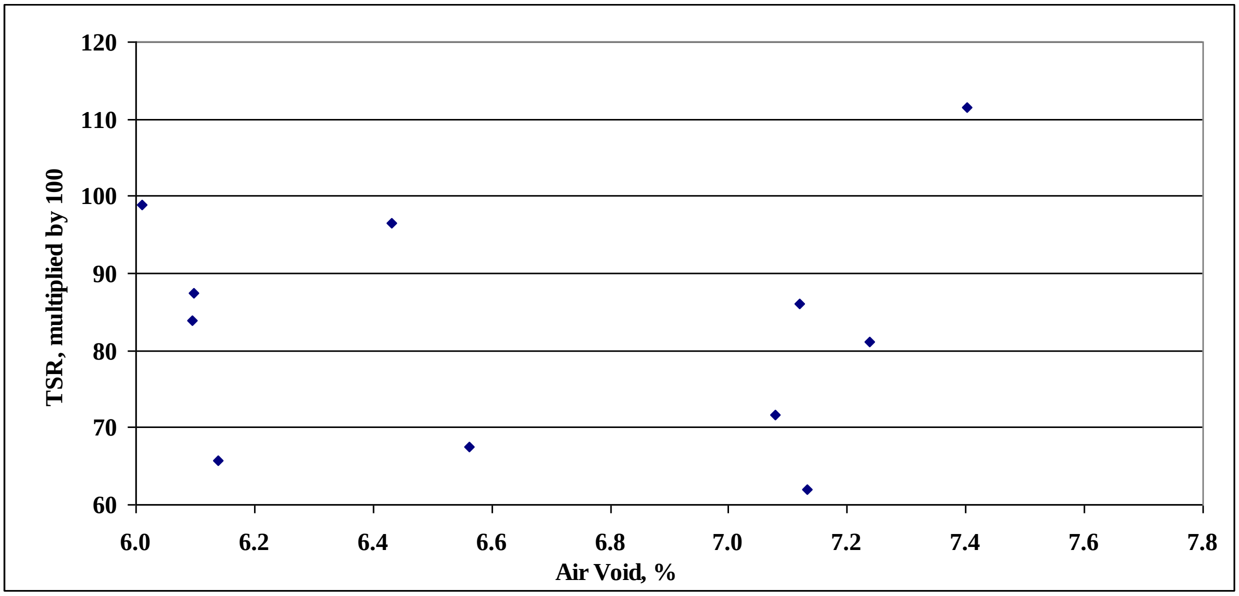

Figure 19 Correlation between the Air Voids of the Conditioned Mixes and the Resulting TSR

RESULTS OF TESTS WITH MMLS3

Testing with the Model Mobile Load Simulator 3rd Scale was conducted as planned except that an additional mix was included in the testing. Originally, only control mixes of TRC and CLA were considered. Later, the control mix of HBC was added to the group. However, the original mixes (TRC and CLA) were tested at three replicate levels but the added mix (HBC) was tested as an individual specimen. The test briquettes were prepared out of SGC specimens, as explained in Chapter 2. The MMLS3 device accommodates a total of nine specimens, and the first and last specimens were used as dummy specimens to support the entrance and exit of the tracking wheel. Figure 20 shows the sequence of seven specimens used in the MMLS3 track for both dry and wet testing. Wet specimens were tested to 400,000 cycles while dry specimens were subject to 800,000 cycles. Details of testing are provided in Chapter 2.

MMLS3 - WET TEST

Test Bed Configuration

7	6	5	4	3	2	1
CLM1 6.73% AV	CLM2 6.49% AV	TRM-T2 7.77% AV	CLM-T1 6.56% AV	HBC-T1 7.09% AV	TRM1 7.40% AV	TRM2 7.61% AV

Traffic Direction

MMLS3 – DRY TEST

7	6	5	4	3	2	1
CLM2	CLM1	TRM3	CLM3	HBM1	TRM2	TRM1
6.17% AV	6.21%AV	7.32%AV	6.28%AV	6.50%AV	7.79%AV	7.89%AV

Traffic Direction

Figure 20 Sequence of Specimens in Both Wet and Dry Conditions for MMLS3 Testing

Tables 14 and 15 and Figure 21 provide results for various specimens for dry and conditioned (wet) testing, respectively. Comparison of permanent deformations from dry and wet specimens is presented in Figures 22 through 24, while a general comparison of results is presented in Figure 25. It can be observed that the total rutting for dry specimens ranges between 2 and 3.5 mm, while for the wet specimens, the range is 2.5 to 4 mm. The HBC mix appears to have the best moisture damage resistance compared with the other two mixes, although care should be taken in drawing definite conclusions in this regard, since only one specimen of the HBC mix was tested compared with three specimens for each of the other two mixes.

MMLS3 – DRY TEST

Table 14 Rutting Levels from MMLS3 for Dry Testing

Sequence	Specimen ID	Aggregate Source	Air Voids %	Average Rutting (mm) after 400,000 Cycles	Average Rutting (mm) after 800,000 Cycles
1	TRM1	Troy sand and gravel	7.9	3.16	3.44
2	TRM2	Troy sand and gravel	7.8	2.33	2.66
3	HBM1	Hasbrouck sand and gravel	6.5	2.96	3.54
4	CLM3	Conneaut sand and gravel	6.3	2.63	3.00
5	TRM3	Troy sand and gravel	7.3	2.33	2.62
6	CLM1	Conneaut sand and gravel	6.2	1.80	2.09
7	CLM2	Conneaut sand and gravel	6.2	2.38	2.83

Table 15 Rutting Levels for MMLS3 for Wet Testing

Sequence	Specimen ID	Aggregate Source	Air Voids %	Average Rutting (mm) after 400,000 Cycles
1	TRM2	Troy sand and gravel	7.6	4.04
2	TRM1	Troy sand and gravel	7.4	3.40
3	HBC-T1	Hasbrouck sand and gravel	7.1	3.86
4	CLM-T1	Conneaut sand and gravel	6.6	3.69
5	TRM-T2	Troy sand and gravel	7.8	3.32
6	CLM2	Conneaut sand and gravel	6.5	3.34
7	CLM1	Conneaut sand and gravel	6.7	2.42

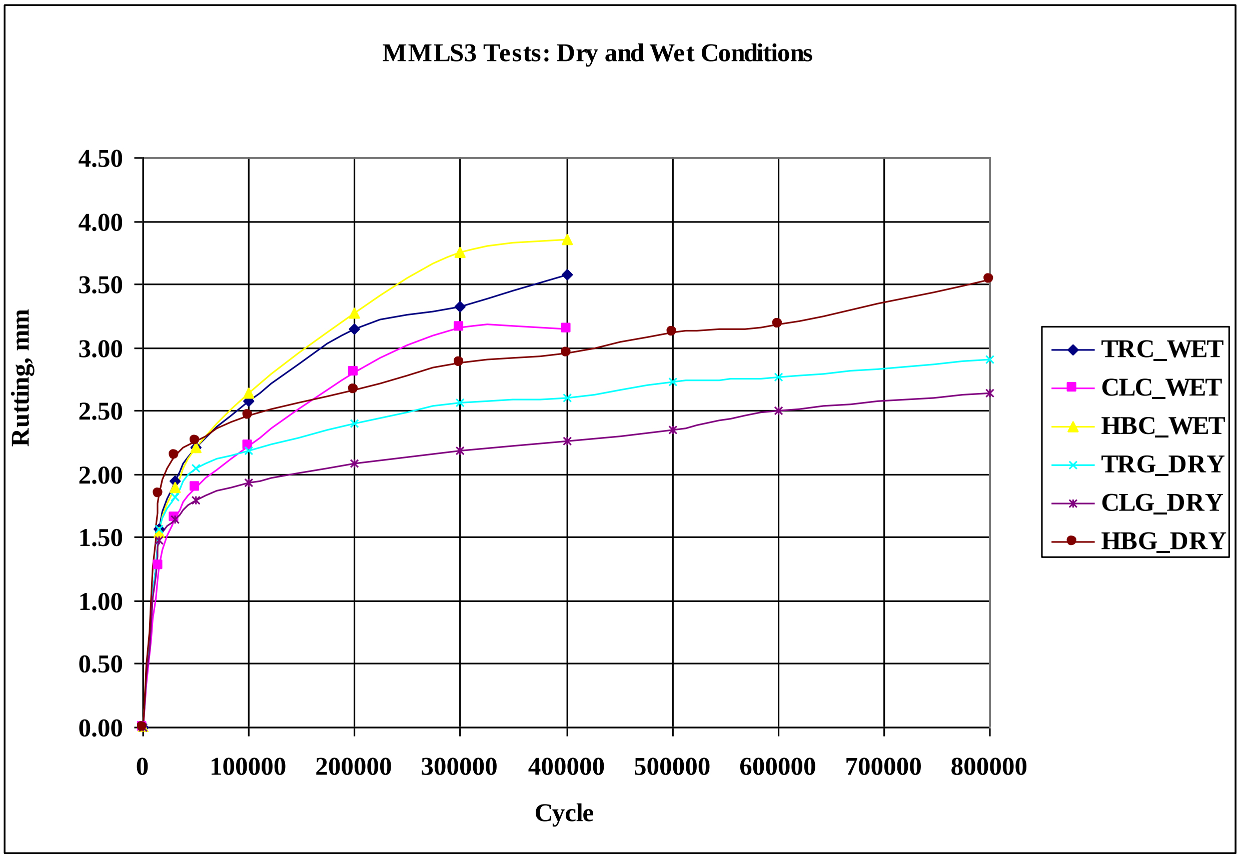

Figure 21 Rutting as a Function of MMLS3 Cycles for Different Mixes

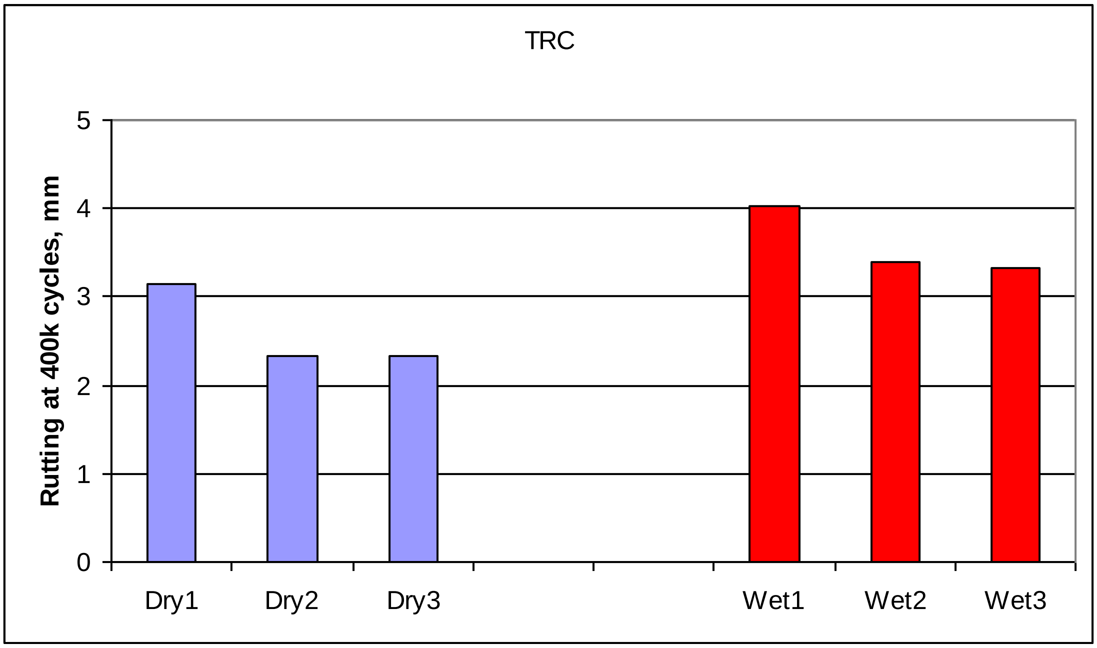

Figure 22 Rutting of Dry versus Conditioned Specimens for the TRC Mix

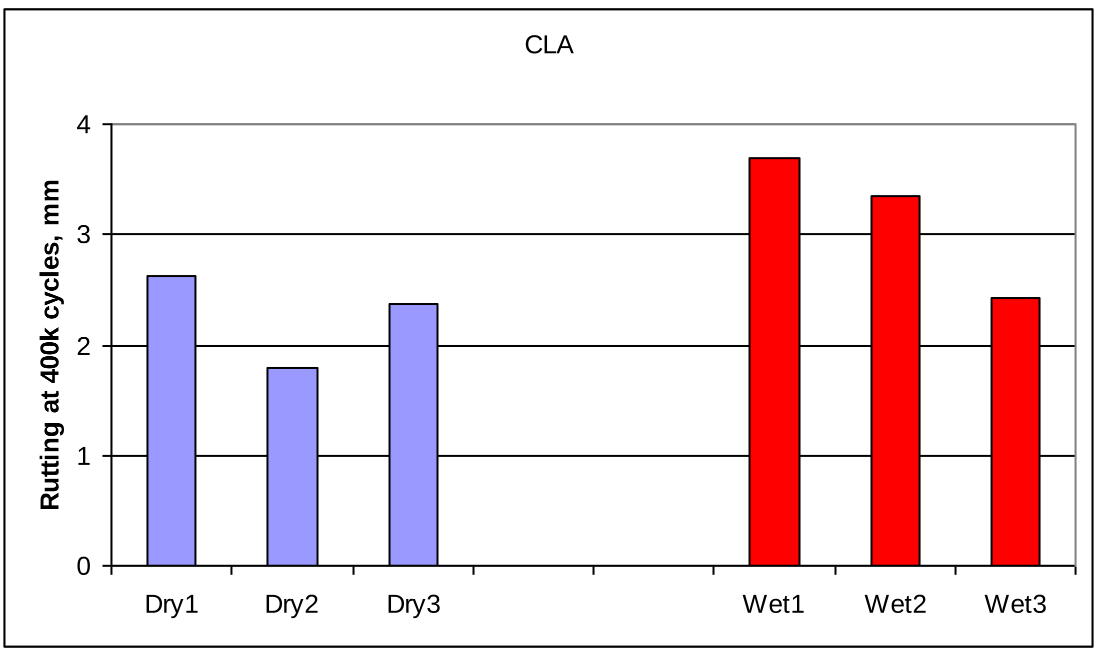

Figure 23 Rutting of Dry versus Conditioned Specimens for the CLA Mix

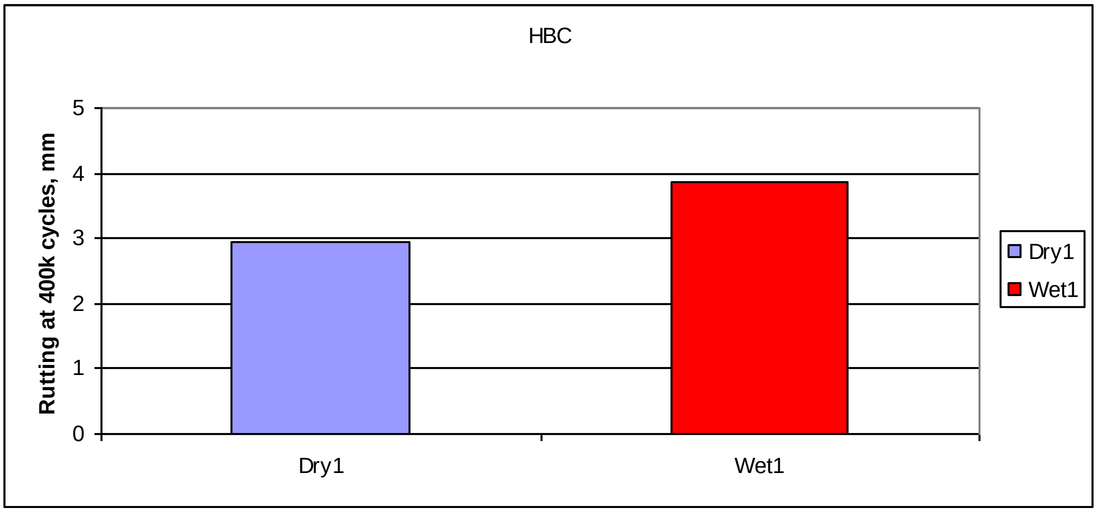

Figure 24 Rutting of Dry versus Conditioned Specimens for the HBC Mix

Figure 25 Average Maximum Rutting for Each Mix after Trafficking 400,000 Cycles

Figure 26 is provided in an attempt to investigate the relationship between indirect tensile strength of dry specimens and the average rutting observed in MMLS3 for each mix. The graph indicates that no relationship exists.

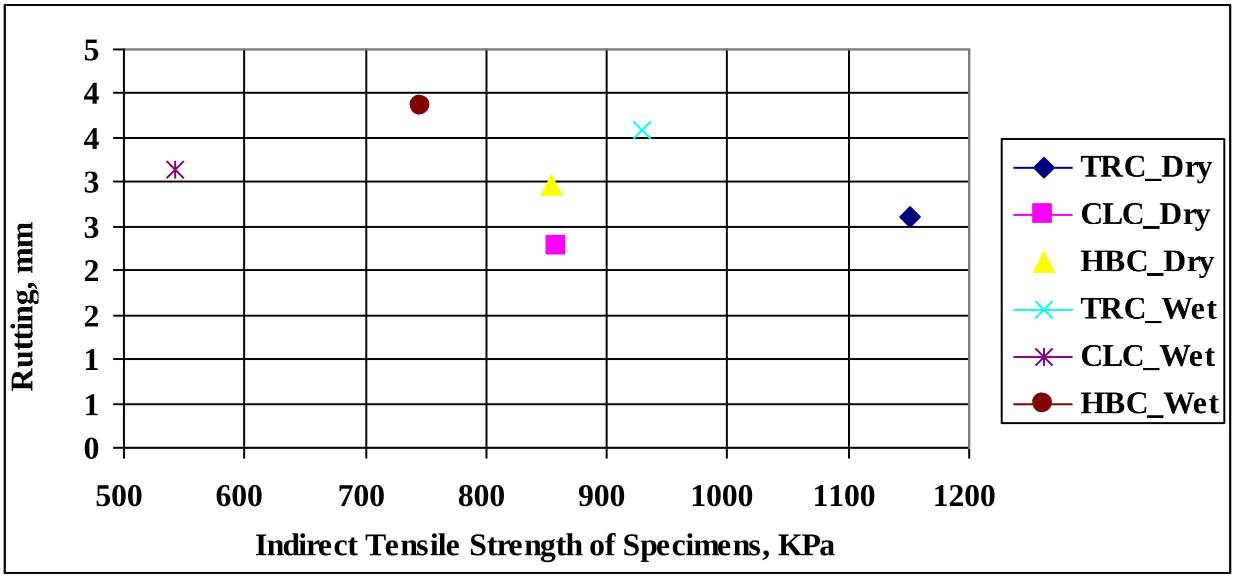

Figure 26 Scatter Plot of Rutting versus Indirect Tensile Strength of Specimens

It was also decided to compare the average rutting in MMLS3 versus the tensile strength ratio (TSR). For this purpose, Table 16 was developed. In this table, the term ratio refers to the average permanent deformation for wet specimens over the average permanent deformation for dry specimens, similar to the concept of tensile strength ratio, which is determined based on the ratio of average strength of dry specimens over that of conditioned specimens. The relationship between rutting ratio and TSR is presented in Figure 27. It seems that the high TSR mix (HBC) has the lowest rutting ratio (approximately 1.3). While the data are still insufficient, this relationship is consistent with the expectation that mixes with better moisture damage resistance should have a lower rutting ratio in MMLS3. It may be concluded based on these limited data that mixes with rutting ratios not exceeding 1.3 are moisture damage resistant.

Table 16 Average Rutting for Dry and Wet Mixes and the Corresponding Ratio

Mix	Rutting (mm)		Ratio (Wet/Dry)
	Dry Mix	Wet Mix	
TRC	2.60	3.58	1.38
CLC	2.27	3.15	1.39
HBC	2.96	3.86	1.31

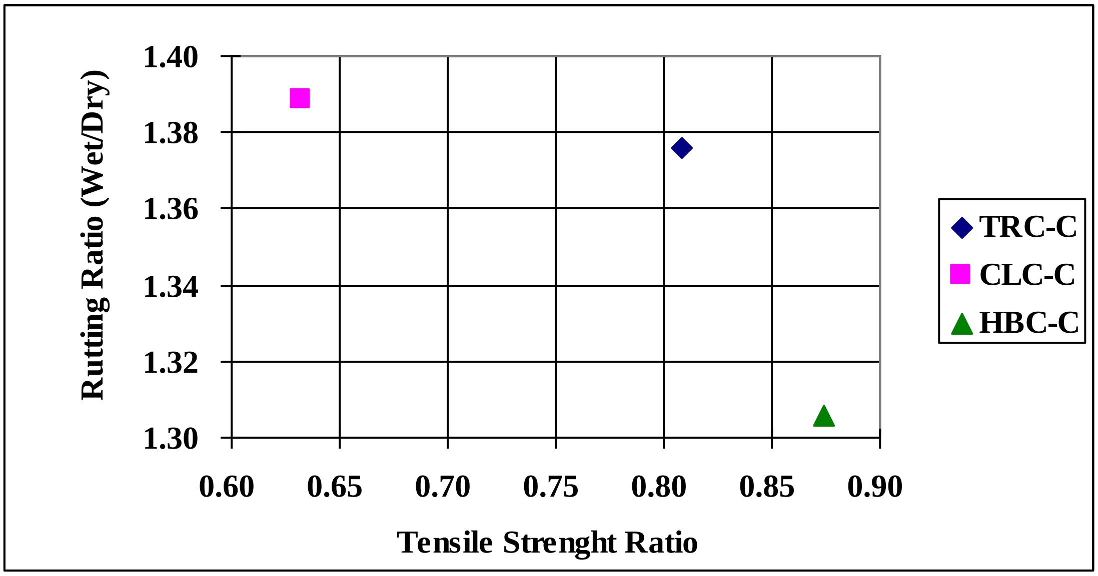

Figure 27 Scatter Plot of Rutting Ratio versus TSR

Interpretation of MMLS3 Test Results and Correlation with Field Performance

An important question to address is the relationship between the results from MMLS3 tests and actual field performance of these mixes. A thorough investigation of the literature reveals a considerable number of research papers and reports on the use of MMLS3 in testing asphalt concrete specimens. However, there is little information on how the MMLS3 results could be extended to the field performance of the tested mixtures. Perhaps part of the reason for such a challenge in finding proper correlation is that field performance is highly dependent on the pavement structure in terms of layer thickness and underlying foundation. Field performance is also highly affected by temperature variations at the site, whereas MMLS3 testing is maintained at a constant temperature. In spite of these differences, it is highly desirable to determine the relationship between MMLS3 test results and expected pavement performance, even if such relationship might be crude and approximate. Therefore, an attempt was made in this research to find the answer.

First, a power law equation was used to project rutting level in MMLS3 testing for each of the mixes. This equation is presented in the following form:

$$\delta = a \times N^b \qquad \textbf{(5)}$$

where δ is the rut depth, N is the number of MMLS3 load cycles, and a and b are the constants. For each mix, a and b were determined based on data available for 400,000 cycles of loading. These a

and *b* constants, presented in Table 17, were used to predict the rutting levels, also presented in Table 17. It can be observed that there is an excellent match between predicted rutting and observed rutting, validating the developed power law equation for each mix. Using these equations shows that calculated rut depth at 800,000 cycles matched observed values for dry mixes. The equations were further used to project rutting levels in MMLS3 testing after 10 million cycles, as shown in Table 17. The results indicate that even at this level, the maximum rut depth does not exceed 10 mm, assuming that during these excessive cycles the mix will not become unstable because of getting into tertiary creep mode. Figure 28 also shows how the predicted results compare with measured values for one of the mixes, and it can be seen that an excellent relationship exists. The relationship for other mixes is very similar in terms of the match between predicted and measured values.

Table 17 Projected Rut Depths in MMLS3 Track Using Power Law

MMLS3 Cycles N x 1,000		**TRC-Wet**	**CLC-Wet**	**HBC-Wet**	**TRC-Dry**	**CLC-Dry**	**HBC-Dry**
	a	**0.1404**	**0.0852**	**0.0982**	**0.3891**	**0.4510**	**0.5320**
	b	**0.2510**	**0.2863**	**0.2871**	**0.1495**	**0.1239**	**0.1330**
0		0.00	0.00	0.00	0.00	0.00	0.00
15		1.57	1.34	1.55	1.64	1.48	1.91
30		1.87	1.63	1.89	1.82	1.62	2.10
50		2.12	1.89	2.19	1.96	1.72	2.24
100		2.53	2.30	2.68	2.17	1.88	2.46
200		3.01	2.81	3.27	2.41	2.05	2.70
300		3.33	3.15	3.67	2.56	2.15	2.85
400		3.58	3.42	3.99	2.68	2.23	2.96
500		3.78	3.65	4.25	2.77	2.29	3.05
600		3.96	3.85	4.48	2.84	2.34	3.12
800		4.26	4.18	4.86	2.97	2.43	3.24
10,000		8.02	8.61	10.04	4.33	3.32	4.54

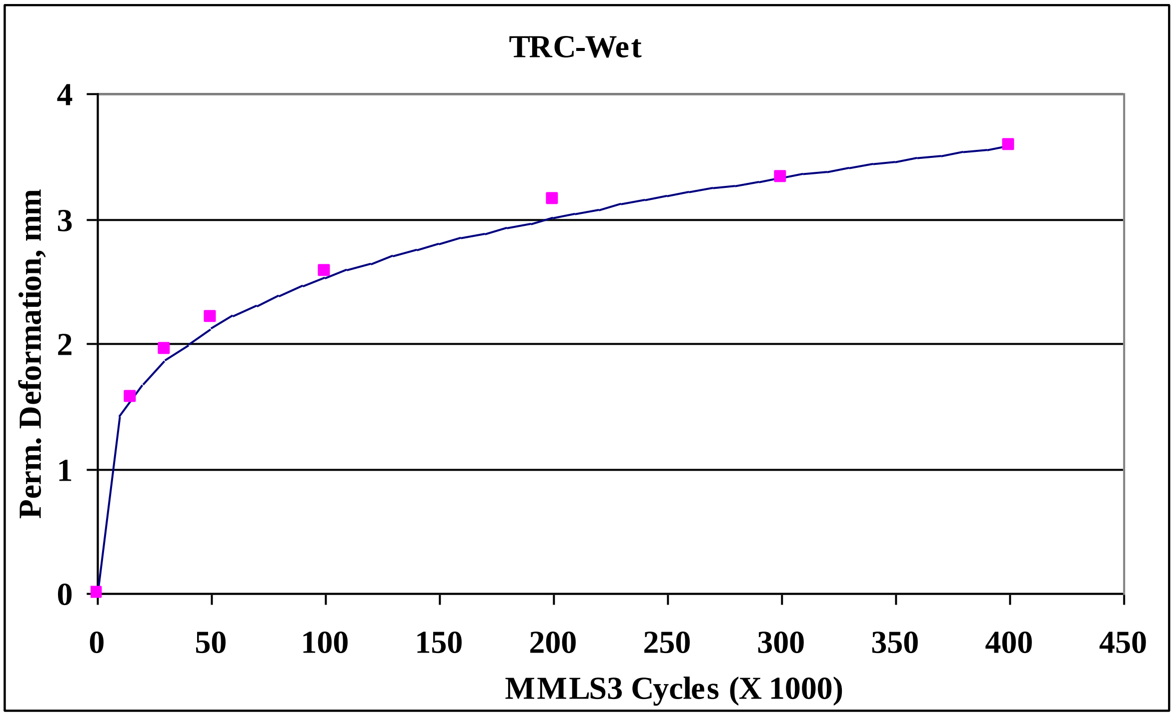

Figure 28 Comparison of Power Law Predicted Deformation and Measured Deformation

The next step is extending these results to 18-kip equivalent single axle loads (ESALs), and that is where no such relationship could be found in the literature. Work by Kumar (2006) using the *Mechanistic-Empirical Pavement Design Guide* (MEPDG) provided a value between 2.65 and 3.31 for the equivalent wheel load factor (EWLF) for MMLS3, with the value depending on the stiffness of the underlying layer. EWLF is defined as

$$ELWLF_B = \frac{(N_f)_{18}}{(N_f)_B} \quad \textbf{(7)}$$

where $(N_f)_{18}$ and $(N_f)_B$ refer to cycles to failure under 18-kip truck axles and under MMLS loading, both for the model scaled pavement. Even if, conservatively, a ratio of 2 is selected for ELWF, one still finds that the number of ESALs for MMLS3 testing could be twice the number of MMLS3 trafficking cycles. Of course, this is for the model case, and further work is needed to draw a more reliable conclusion on the relationship between the observed MMLS3 deformation and expected field performance.

It is the authors' conclusion that moisture has influenced the level of permanent deformation observed in MMLS3 testing. However, it remains to be determined how significant this impact is and how this relates with expected field performance. It is, however, clear that, based on the MMLS3 data available from other sources, the rutting levels observed in our testing are not excessive and are within the range of properly designed mixes tested by MMLS3 under other projects, such as those conducted by Smit et al. (2002 and 2004).

RESULTS OF DYNAMIC MODULUS TESTING

Modulus of asphalt concrete mixes has been considered among the most important properties measured for this material. Results from dynamic modulus (DM) testing provide the fundamental material properties that are used with the newly developed *Mechanistic Empirical Pavement Design Guide*. Therefore, through coordination with the project technical advisor it was decided to explore the potential of this test in discriminating the behavior of poorly performing mixes from well-performing mixes in regard to moisture damage. In the following sections, mixes included in DM testing, conditioning of the mixes for DM testing, and the test results are presented and discussed.

Mixes and Specimens for DM Testing

Dynamic modulus testing was not a major portion of this research and therefore did not include all mixes. Two of the five mixes researched in this study, Troy Sand and Gravel and Conneaut Lake Gravel, were considered for DM testing. For these two mixes, three replicate specimens were prepared at 7 percent air void from gyratory-compacted specimens according to the procedure explained in Chapter 2.

Testing Approach

Details of dynamic modulus testing were explained in Chapter 2. DM tests were conducted on the same specimen three times. The first test was on the dry, unconditioned specimen, followed by a second test after the specimen was exposed to one complete cycle of water conditioning, and a third test after a 2nd cycle of water conditioning. Testing was conducted at a temperature of 25°C and loading frequencies of 10, 5, 2, and 1 Hz. The sequence of frequencies was from highest to lowest to minimize damage due to loading. Furthermore, attempts were made to apply a load level that would induce a strain level between 30 and 70 microstrains to minimize specimen permanent deformation.

Conditioning Approach

The idea behind this testing was to conduct the same conditioning approach that was used in AASHTO T 283 test method with the exception of conditioning time; that is, using a 30-minute conditioning time rather than targeting a specified level of degree of saturation. The combined conditioning-DM tests were conducted to determine the impact of the water conditioning on dynamic modulus of the mix. After each specimen was tested for dynamic modulus at four frequencies, it was exposed to water conditioning as follows.

- 30 minute vacuum at partial pressure of 26 inches of Hg
- 16 hours of -18°C freeze
- 24 hours of 60°C water bath
- 2 hours of conditioning at 25°C

The preceding sequence completes one cycle of conditioning. Afterwards, the specimen was subjected to a second set of dynamic modulus tests at four frequencies. This was followed by the second cycle of water conditioning for 2 days, as explained above, before the final dynamic modulus testing was conducted.

DM Test Results

Details of DM testing are provided in Appendix C. Details include load levels, strain levels, and resulting modulus at each frequency for each specimen and at different conditioning levels. A summary of results is provided in Table 18. The shaded areas indicate unreasonable results or outliers which were not included in computation of averages. It can be seen that, as expected, the modulus drops as loading frequency decreases (Figures 29 and 30). For TRC mix, the modulus at 10 Hz is approximately 4,200 MPa (600,000 psi) while it is approximately 1,900 MPa (270,000 psi) at a loading frequency of 1 Hz. More importantly, in general, a drop in modulus is observed for all specimens and at all frequencies after water conditioning. For example, at 10 Hz frequency for the TRC mix, the specimen modulus drops from approximately 4,200 MPa to 3,000 MPa once it goes through a complete cycle of water conditioning. Figures 29 and 30 also show the level of modulus drop for different conditioning cycles. It is quite obvious that as the number of conditioning cycles increases, the modulus decreases.

Table 18. Summary of Results from DM Tests

Mix	Frequency	Dynamic Modulus, Mpa			Moduli Ratio	
	Hz	Mod. 1	Mod. 2	Mod. 3	DMR-1	DMR-2
TRC-1	10	4195.6	2724.5	2402.8	0.65	0.57
	5	3148.5	1986.0	1703.6	0.63	0.54
	2	2243.2	1447.0	1246.9	0.65	0.56
	1	1635.4	1263.1	999.7	0.77	0.61
TRC-2	10	NA	3416.9	2843.6	NA	NA
	5		2696.5	2482.3		
	2		2232.4	1713.2		
	1		1610.7	1318.0		
TRC-3	10	4265.2	2813.5	3495.1	0.66	0.82
	5	3103.7	1994.6	2729.8	0.64	0.88
	2	2155.8	2279.9	2368.5	1.06	1.10
	1	2104.8	1443.1	1959.8	0.69	0.93
AVERAGE of All 3 for TRC	**10**	**4230.4**	**2985.0**	**2623.2**	**0.65**	**0.57**
	5	**3126.1**	**2225.7**	**2092.9**	**0.64**	**0.54**
	2	**2199.5**	**1986.4**	**1480.1**	**0.65**	**0.56**
	1	**1870.1**	**1439.0**	**1158.8**	**0.73**	**0.61**
CLA-1	10	3211.4	2948.0	2108.6	0.92	0.66
	5	2692.4	2024.4	1648.3	0.75	0.61
	2	2018.9	1557.8	1210.2	0.77	0.60
	1	1833.4	1385.0	1263.1	0.76	0.69
CLA-2	10	3491.1	2745.9	2916.4	0.79	0.84
	5	2877.4	2543.5	2391.0	0.88	0.83
	2	2176.5	2084.4	1980.4	0.96	0.91
	1	1953.0	1851.1	1400.4	0.95	0.72
CLA-3	10	4276.5	3258.2	2701.4	0.76	0.63
	5	2936.5	2594.4	2142.7	0.88	0.73
	2	2359.9	2045.4	1695.4	0.87	0.72
	1	2149.2	1981.9	1488.7	0.92	0.69
AVERAGE of All 3 for CLA	**10**	**3659.7**	**2984.1**	**2512.5**	**0.82**	**0.66**
	5	**2835.5**	**2387.4**	**2019.6**	**0.84**	**0.61**
	2	**2185.1**	**1895.9**	**1595.3**	**0.77**	**0.60**
	1	**1978.5**	**1739.3**	**1331.8**	**0.88**	**0.69**

NOTES: TRC: Troy Sand and Gravel
DMR-1: Ratio of Modulus after 1^{st} Cycle Conditioning to Unconditioned Modulus.
DMR-2: Ratio of Modulus after 2^{nd} Cycle Conditioning to Unconditioned Modulus.
Shaded Cells contain outliers or unreliable data and were not included in calculation of averages.
MOD1, MOD2, and MOD3 refer to modulus of specimen at dry condition, after 1st cycle and after2nd cycle conditioning

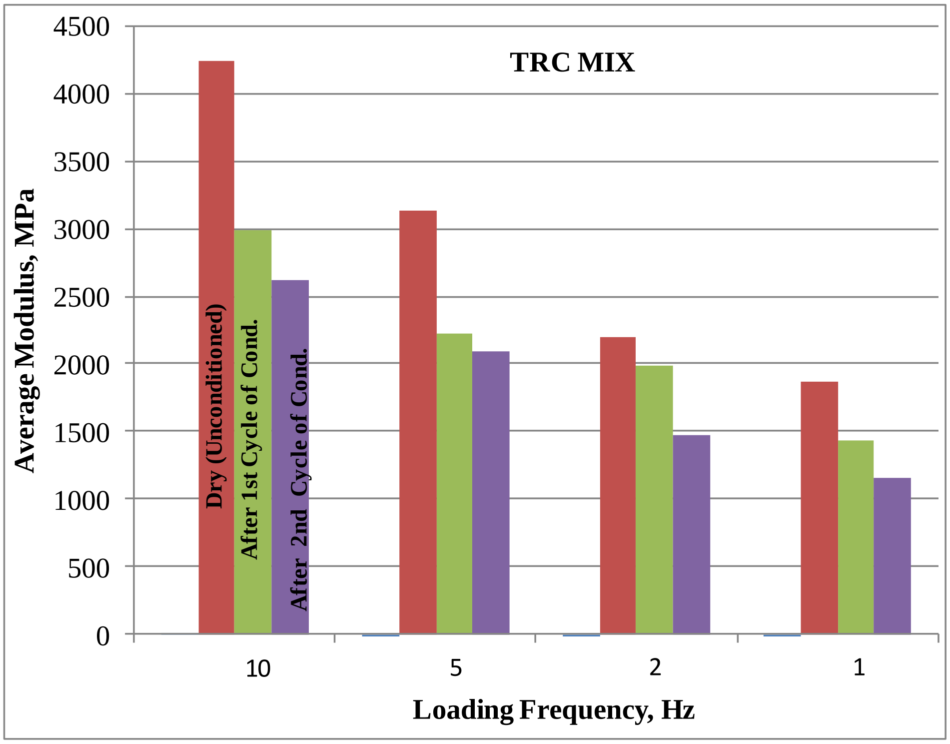

Figure 29 Average Modulus for Different Frequencies and Different Conditioning Levels for the Troy Sand and Gravel Mix

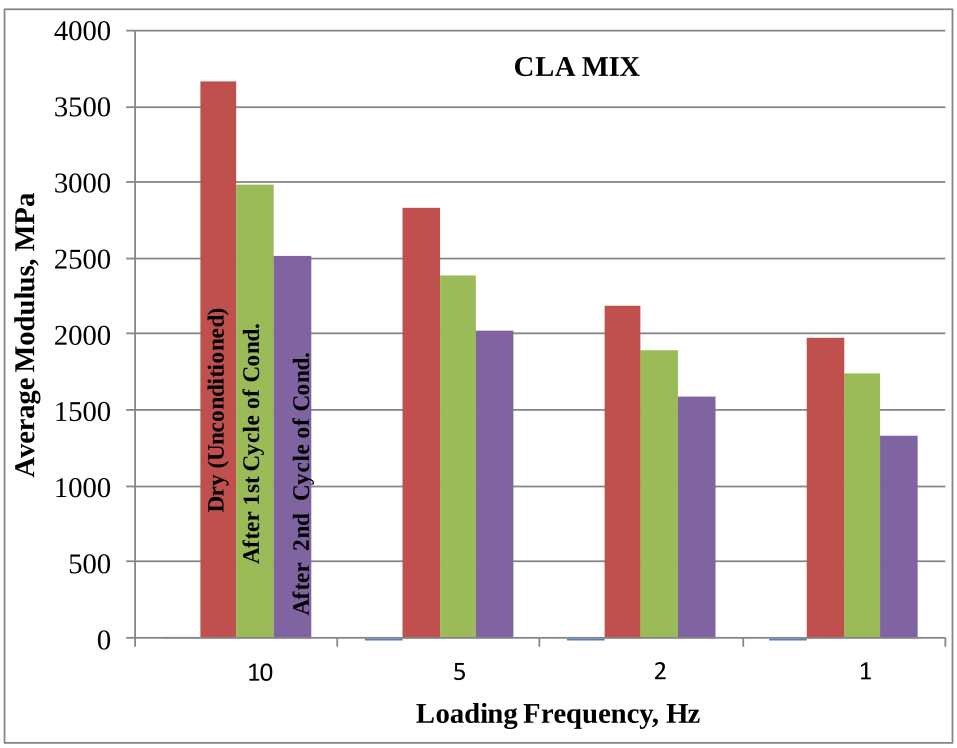

Figure 30 Average Modulus for Different Frequencies and Different Conditioning Levels for the Conneaut Lake Mix

The most important result from this test is the ratio of the modulus of a fully conditioned specimen to the modulus of that specimen in dry condition, as presented in Figures 31 and 32 for the TRC and CLA mixes, respectively. The TRC mix, on average, retained 65 percent of its original modulus value at the 10-Hz frequency after the first conditioning cycle. This retainage level is based on the definition for the ratio of moduli as shown in equation 8. According to this definition, for the TRC mix the ratio of moduli is approximately 0.35.

Ratio of Moduli = (Dyn. Modulus After Cond.)/ (Dyn. Modulus Before Cond) ***(8)***

While the modulus drops after conditioning for all frequencies, there is not a significant difference in ratio of moduli for different loading frequencies. Overall, it is suggested that more than 30 percent drop in modulus as a result of water conditioning is significant. Figure 32 indicates that the ratio of moduli for the CLA mix after one conditioning cycle is about 0.82, implying a better retainage of modulus compared with the TRC mix. This observation is not consistent with the results obtained from T 283 tests, in which it was found that the TRC mix had a higher TSR value compared with the CLA mix. However, the results indicate that after two cycles of water conditioning, there is a significant drop of modulus for both mixes, indicating susceptibility of the mixes to moisture damage. It should be noted that the results presented here for dynamic modulus are for control mixes; that is, mixes without any liquid antistripping agent, lime, or any other modifications.

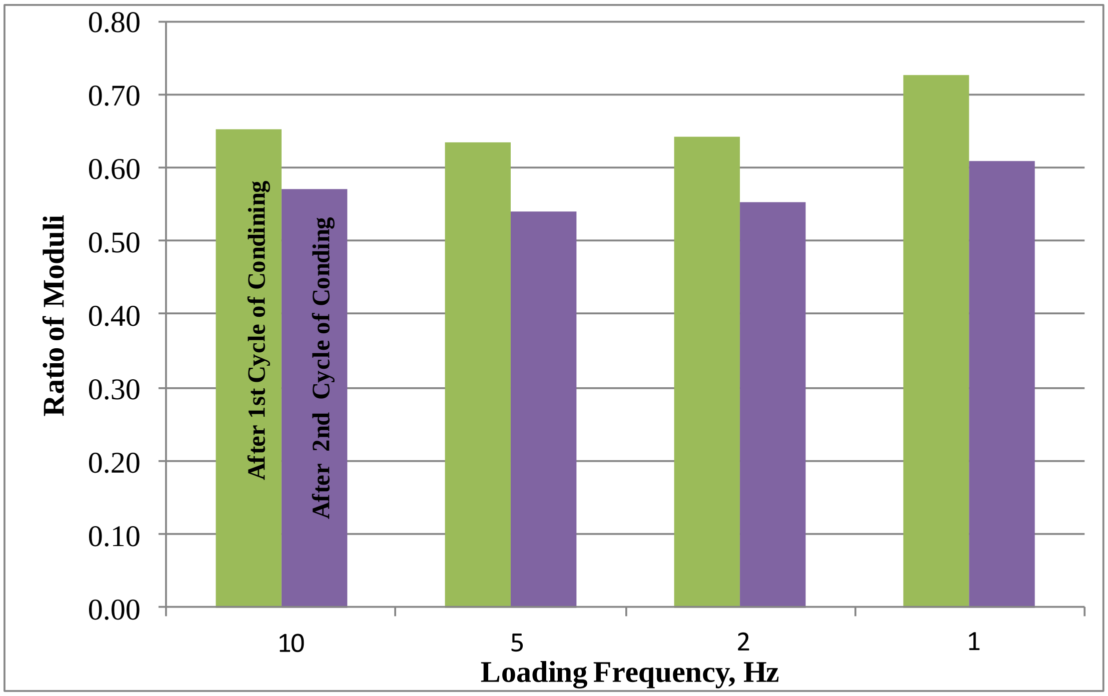

Figure 31 Ratio of Moduli for Different Frequencies (Troy Sand and Gravel)

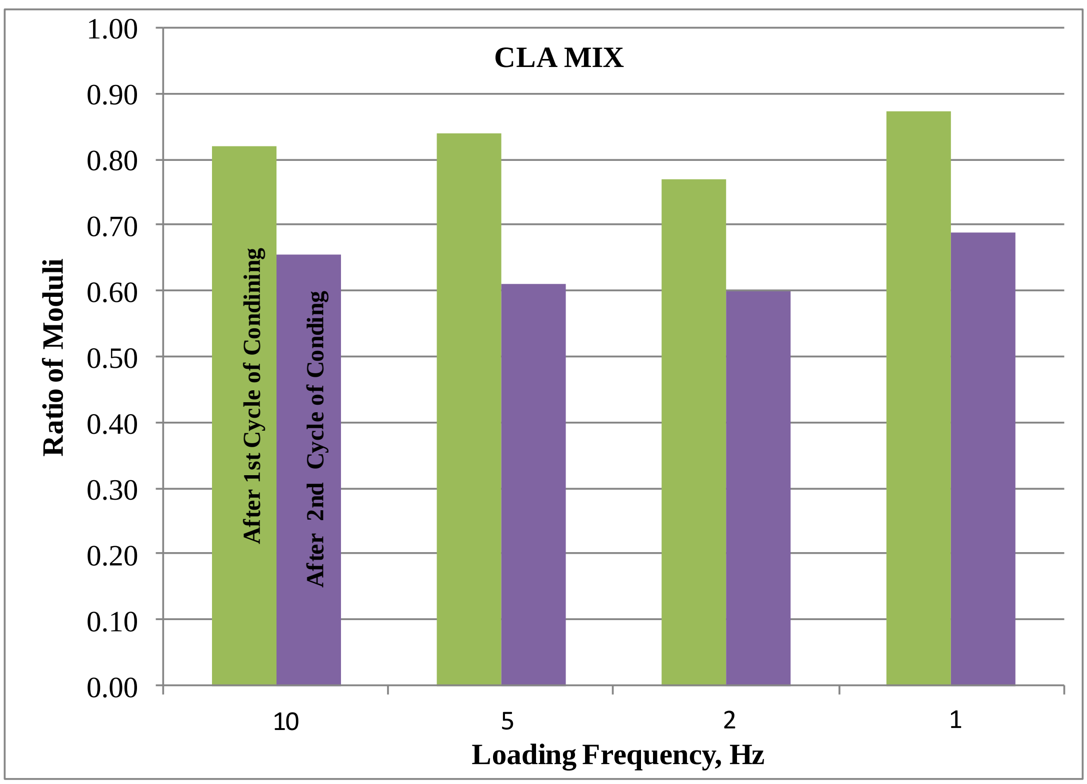

Figure 32 Ratio of Moduli for Different Frequencies (Conneaut Lake)

CHAPTER FOUR
SUMMARY AND CONCLUSIONS

The results presented in this report demonstrate the outcome of a 2-year laboratory study by the Northeast Center of Excellence for Pavement Technology (NECEPT) at The Thomas D. Larson Pennsylvania Transportation Institute on moisture damage of asphalt concrete using PennDOT District 1-0 local aggregates. The study was sponsored by the Pennsylvania Department of Transportation and the Mid-Atlantic Universities Transportation Center. This study was initiated to investigate the possibility of using local aggregates, those not satisfying the criteria for Type A material, in hot-mix asphalt concrete pavements.

District 1-0 has been faced with a shortage of Type A aggregate. Most of the local aggregates satisfy specification requirements for Type A aggregate, except requirements for sodium sulfate soundness and water absorption. A detailed laboratory evaluation was conducted and a series of tests were performed on laboratory-prepared specimens using these aggregates. Five gravel aggregates (four Type C and one Type A) were included in the study. For each gravel source, a control mix was considered, along with three additional mixes, prepared by liquid antistripping agent, hydrated lime, and 50/50 blend with a limestone aggregate with a known history of good performance.

The tests included the PennDOT-modified version of AASHTO Test Method T283 (Tensile Strength Ratio), Model Mobile Load Simulator 3rd Scale, and dynamic modulus tests after repeated freeze-thaw cycles.

From moisture damage evaluation using tensile strength ratio it was observed that TSR for all control mixes of these gravel sources, except the TRC and HBC aggregates, were less than 0.80, indicating susceptibility of these mixes to moisture damage. It was also concluded that the liquid antistripping agent made a significant improvement in TSR value and therefore in resistance to moisture damage. In summary, for all aggregates TSR is increased when LAS is used, and for two of the failing sources, LAS increases TSR to a level exceeding 0.80, making the mix an acceptable one. The only mix that still remains in the failing range after addition of LAS is the IAC source, even though TSR is increased as a result of adding LAS. Testing mixes with the 50/50 blend of gravel and limestone for the #8 portion of the aggregate delivered mixed results in terms of its impact on TSR. Using such a blend yielded an increase in TSR values (improvement in moisture damage resistance) for TRC, LLC, and CLA aggregates, although the level of improvement observed using the 50/50 blend was lower than that observed using LAS. Except for one mix, using hydrated lime did not improve the tensile strength ratio of the mixes. It is well known that use of hydrated lime with siliceous gravel aggregates, in general, results in improvement of moisture damage resistance. The reasons why such behavior was not observed for the mixes used in this study is unknown at this time. The poor performance of lime treated mixes of this study could have come from the impact of the interaction between the fine material in the mix and the lime as the same fine material was used for all mixes. However, further investigation is needed to truly identify why adding lime did not improve moisture damage resistance.

Testing with MMLS3 provided valuable information. Only three control mixes were included in this part of the study, and testing was conducted under both dry and wet conditions. In the wet condition, specimens were immersed in water while being tracked by the wheels. This testing was conducted for 400,000 cycles of wheel loading at 52 °C. It was observed that the total rutting for dry specimens ranged between 2 and 3.5-mm, while for the wet specimens, the range was 2.5 to 4-mm. The HBC mix appeared to have the best moisture damage resistance compared with the other two mixes, although care should be taken in drawing definite conclusions in this regard, since only one specimen of the HBC mix was tested compared with three specimens for each of the other two mixes.

Dynamic modulus tests were conducted on specimens of two of the mixes without any treatment (Troy Sand and Gravel and Conneaut Lake Gravel). Dynamic modulus tests were conducted on dry specimens followed by further testing on the same specimen after exposure to two cycles of conditioning. Significant drop of modulus was observed for the Troy Sand and Gravel mix after the first cycle of conditioning. Significant drop of modulus was observed for both mixes after the second cycle of conditioning, indicating susceptibility of both mixes to moisture damage if no treatment is applied.

It is the authors' conclusion that moisture influenced the level of permanent deformation observed in the MMLS3 testing, even though the magnitude of this impact is not clear from the data. Based on the MMLS3 data available from other sources, the rutting levels observed in our testing are not excessive and are within the range of properly designed mixes tested by MMLS3 under other projects.

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APPENDIX

A

Results of T283 Tests

Table A-1 Results of AASHTO T283 for Different Mixes

Mix	Dry Condition			Wet Condition				TSR
	Specimen ID	Air Void %	Tensile Str Kpa	Specimen ID	Air Void %	Tensile Str Kpa	Sat Level %	%
TRC Control	TR6	7.2	3017.7	TR7	7.5	897.7	41.1	80.8
	TR8	7.2	1256.2	TR9	7.2	961.7	31.3	
	TR10	7.2	1044.8	TR11	7.3	930.3	30.3	
Average		**7.2**	**1150.5**		**7.3**	**929.9**	**34.2**	
Std Dev		**0.04**	**1083.21**		**0.15**	**32.02**	**5.97**	
COV		**0.51**	**94.15**		**2.04**	**3.44**	**17.43**	
TRC with LAS	TRA1	7.1	832.6	TRA7	7.4	757.7	72.5	111.5
	TRA2	7.3	829.7	TRA8	7.4	952.7	71.9	
	TRA3	7.4	810.7	TRA9	7.3	1046.9	72.3	
Average		**7.3**	**824.3**		**7.4**	**919.1**	**72.2**	
Std Dev		**0.14**	**11.89**		**0.06**	**147.51**	**0.32**	
COV		**1.96**	**1.44**		**0.77**	**16.05**	**0.44**	
TRC with AMA	TRS1	7.2	654.9	TRS4	7.1	540.9	83.5	86.1
	TRS2	7.2	614.3	TRS5	7.2	579.0	86.2	
	TRS3	6.9	775.5	TRS6	7.1	639.7	85.7	
Average		**7.1**	**681.6**		**7.1**	**586.5**	**85.1**	
Std Dev		**0.17**	**83.85**		**0.05**	**49.86**	**1.47**	
COV		**2.39**	**12.30**		**0.67**	**8.50**	**1.73**	
LLC Control	LL1	6.6	644.0	LL4	6.2	437.5	92.5	65.7
	LL2	6.4	704.3	LL5	5.8	485.8	90.2	
	LL3	6.2	799.6	LL6	6.4	487.7	85.8	
Average		**6.4**	**716.0**		**6.1**	**470.3**	**89.5**	
Std Dev		**0.23**	**78.45**		**0.27**	**28.48**	**3.40**	
COV		**3.54**	**10.96**		**4.45**	**6.06**	**3.80**	
LLC with LAS	LLA1	6.4	670.2	LLA4	6.2	516.8	80.8	83.9
	LLA2	6.1	813.4	LLA5	6.0	684.4	78.4	
	LLA3	5.9	855.8	LLA6	6.1	762.4	79.6	
Average		**6.2**	**779.8**		**6.1**	**654.5**	**79.6**	
Std Dev		**0.25**	**97.27**		**0.09**	**125.51**	**1.21**	
COV		**4.01**	**12.47**		**1.51**	**19.18**	**1.52**	
LLC with AMA	LLS1	7.5	546.9	LLS4	7.3	483.7	79.2	81.1
	LLS2	7.4	567.4	LLS5	7.2	511.2	81.6	
	LLS3	7.0	648.7	LLS6	7.2	434.9	77.2	
Average		**7.3**	**587.7**		**7.2**	**476.6**	**79.3**	
Std Dev		**0.26**	**53.85**		**0.07**	**38.60**	**2.19**	

COV		**3.59**	**9.16**		**0.93**	**8.10**	**2.77**	
HBC Control	HB1 HB2 HB3	6.7 6.5 6.3	755.2 923.0 885.8	HB4 HB5 HB6	6.0 6.2 6.1	666.6 673.3 901.4	81.9 82.9 79.5	87.4
Average **Std Dev** **COV**		**6.5** **0.16** **2.45**	**854.6** **88.14** **10.31**		**6.1** **0.12** **1.89**	**747.1** **133.66** **17.89**	**81.4** **1.70** **2.09**	
HBC with LAS	HBA1 HBA2 HBA3	6.8 6.7 6.4	732.5 727.0 816.2	HBA4 HBA5 HBA6	6.4 6.5 6.4	689.7 629.5 878.4	74.3 75.1 78.5	96.6
Average **Std Dev** **COV**		**6.6** **0.23** **3.42**	**758.6** **49.97** **6.59**		**6.4** **0.10** **1.54**	**732.5** **129.85** **17.73**	**76.0** **2.23** **2.94**	
HBC with AMA	HBS1 HBS2 HBS3	7.9 7.2 7.0	535.2 656.3 649.7	HBS4 HBS5 HBS6	7.0 7.2 7.1	437.5 381.0 501.5	90.7 91.2 95.8	71.7
Average **Std Dev** **COV**		**7.4** **0.50** **6.72**	**613.7** **68.07** **11.09**		**7.1** **0.11** **1.60**	**440.0** **60.27** **13.70**	**92.6** **2.85** **3.08**	
IAC Control	IA4 IA5 IAT1	7.0 7.0 6.7	883.7 942.0 832.4	IA1 IA2 IA3	7.1 7.2 6.8	457.4 460.2 521.0	71.4 71.6 74.0	54.1
Average **Std Dev** **COV**		**6.9** **0.15** **2.19**	**886.0** **54.84** **6.19**		**7.0** **0.24** **3.47**	**479.5** **35.96** **7.50**	**72.3** **1.46** **2.02**	
IAC with LAS	IAA4 IAA5 IAA6	7.0 7.0 6.9	706.9 722.1 706.2	IAA1 IAA2 IAA3	7.2 7.3 6.9	415.8 389.2 520.2	74.5 72.9 74.1	62.1
Average **Std Dev** **COV**		**7.0** **0.02** **0.22**	**711.7** **8.99** **1.26**		**7.1** **0.20** **2.77**	**441.7** **69.24** **15.68**	**73.8** **0.83** **1.13**	
IAC with AMA	IAS4 IAS5 IAS6	7.4 7.4 7.4	635.5 701.5 719.2	IAS1 IAS2 IAS3	7.6 7.6 7.4	352.3 332.5 371.1	75.6 75.1 77.6	51.4
Average **Std Dev** **COV**		**7.4** **0.04** **0.59**	**685.4** **44.12** **6.44**		**7.5** **0.15** **1.99**	**352.0** **19.29** **5.48**	**76.1** **1.34** **1.77**	
CLA Control	CL1 CL2 CL3	6.4 6.2 5.9	728.9 849.5 997.4	CL4 CL5 CL6	6.1 5.9 6.0	501.1 507.9 620.4	67.6 73.0 73.3	63.3

Average **Std Dev** **COV**		**6.1** **0.25** **4.15**	**858.6** **134.49** **15.66**		**6.0** **0.10** **1.70**	**543.1** **67.03** **12.34**	**71.3** **3.19** **4.47**	
CLA with LAS	CLA1 CLA2 CLA3	6.3 6.2 6.1	1358.3 1073.3 1444.3	CLA4 CLA5 CLA6	6.1 5.9 6.1	1189.5 1251.4 1390.6	65.3 67.8 72.8	98.9
Average **Std Dev** **COV**		**6.2** **0.13** **2.05**	**1292.0** **194.20** **15.03**		**6.0** **0.13** **2.12**	**1277.2** **103.01** **8.07**	**68.7** **3.85** **5.60**	
CLA with AMA	CLS1 CLS2 CLS3	7.1 6.8 6.5	630.1 730.3 745.9	CLS4 CLS5 CLS6	6.5 6.7 6.5	380.5 485.8 554.7	92.0 92.2 92.4	67.5
Average **Std Dev** **COV**		**6.8** **0.34** **5.01**	**702.1** **62.82** **8.95**		**6.6** **0.10** **1.51**	**473.7** **87.76** **18.53**	**92.2** **0.19** **0.21**	

APPENDIX

B

Detailed Graphs from MMLS3 Testing

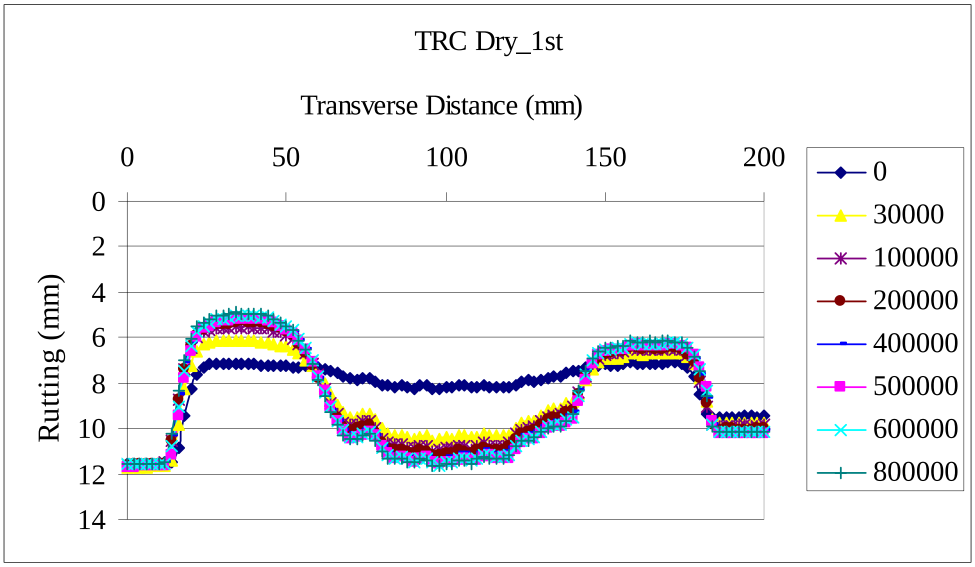

Figure B-1 Rutting versus transverse direction for TRC specimen in the 1st position under dry condition

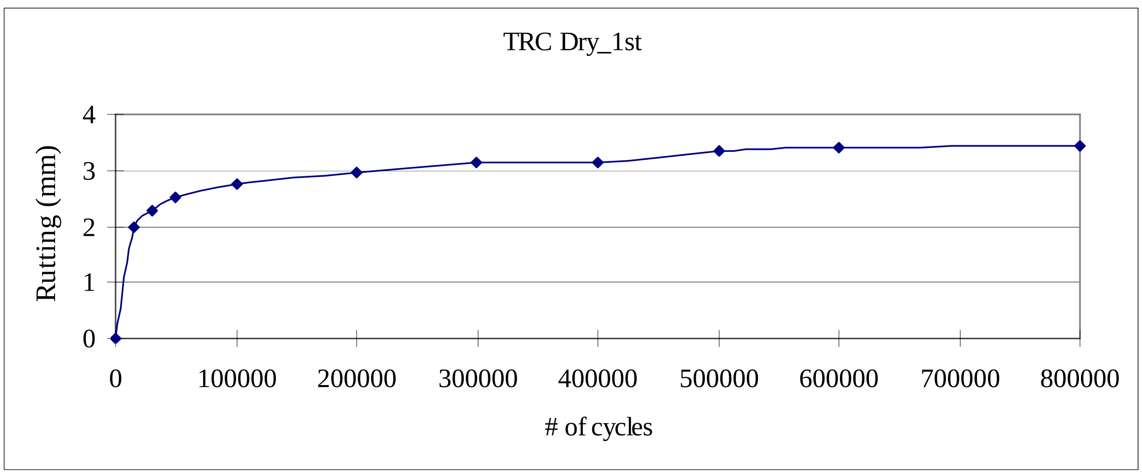

Figure B-2 Rutting versus number of cycles for TRC specimen in the 1st position under dry condition

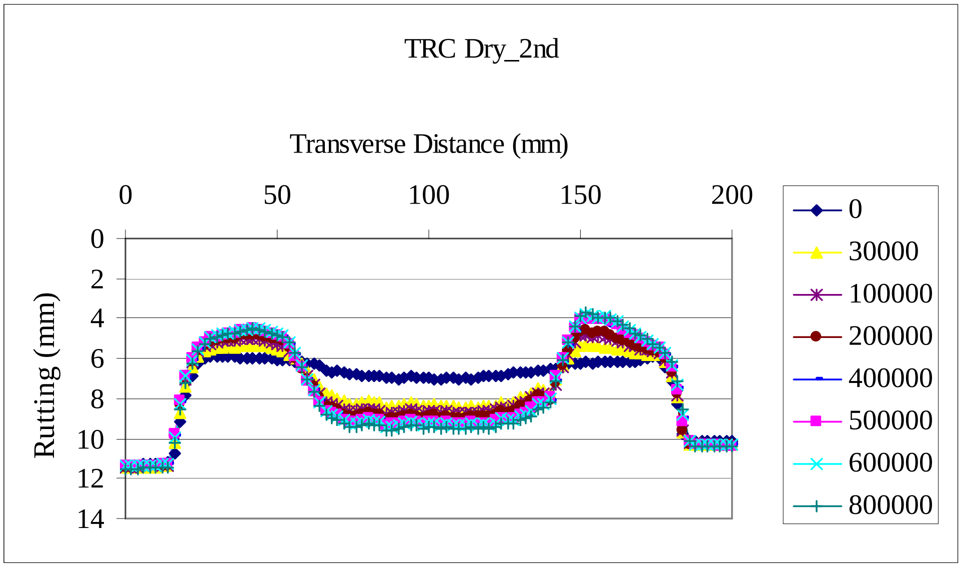

Figure B-3 Rutting versus transverse direction for TRC specimen in the 2nd position under dry condition

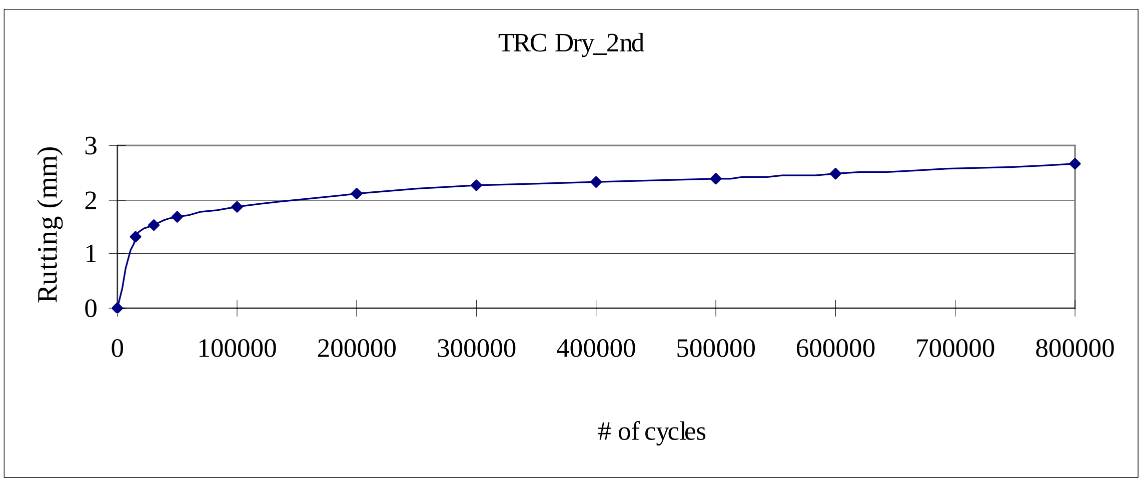

Figure B-4 Rutting versus number of cycles for TRC specimen in the 2nd position under dry condition

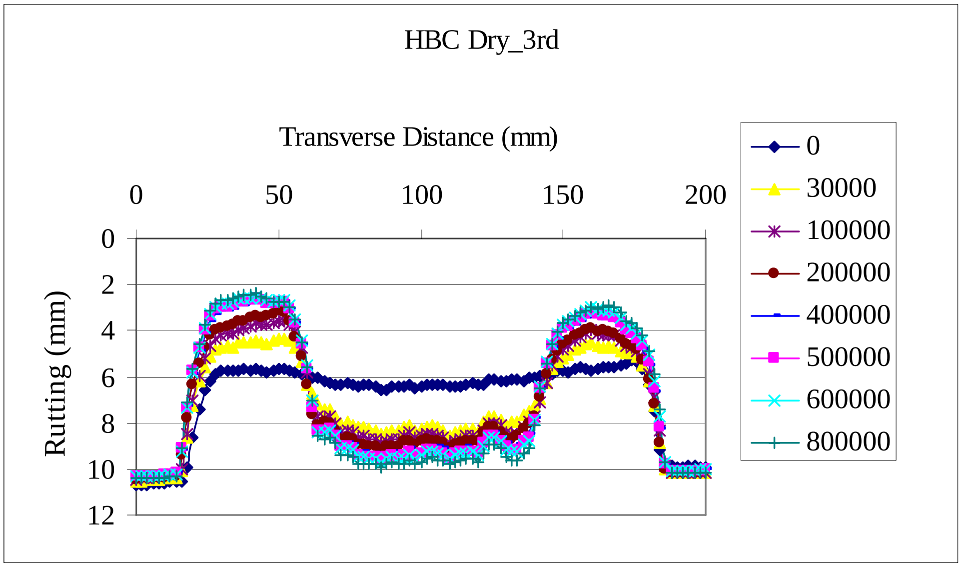

Figure B-5 Rutting versus transverse direction for HBC specimen in the 3rd position under dry condition

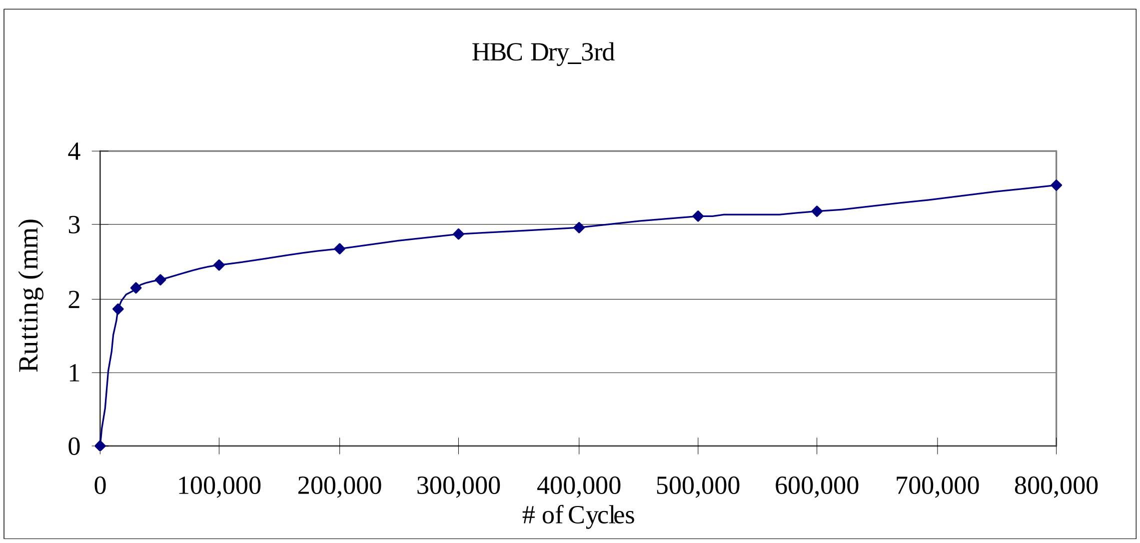

Figure B-6 Rutting versus number of cycles for HBC specimen in the 3rd position under dry condition

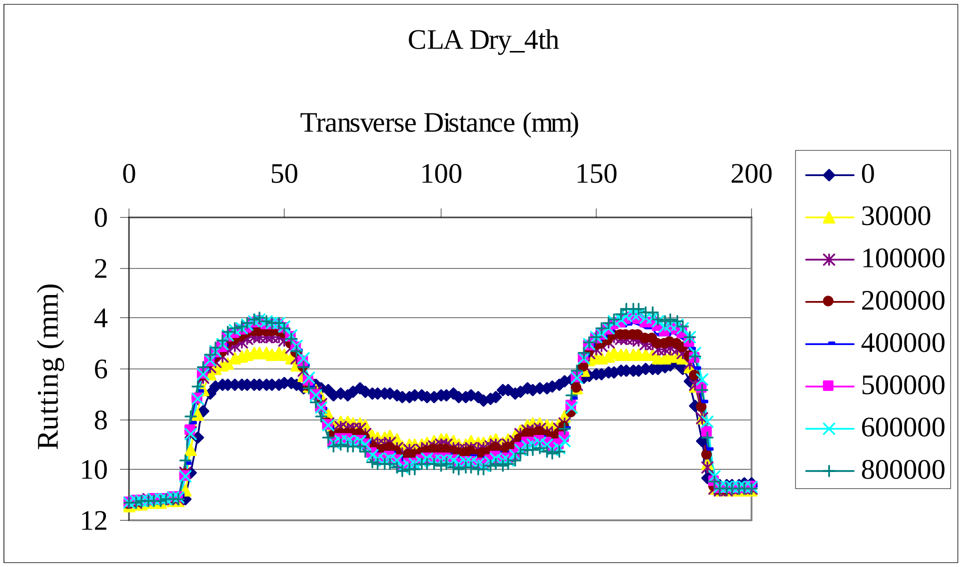

Figure B-7 Rutting versus transverse direction for CLA specimen in the 4th position under dry condition

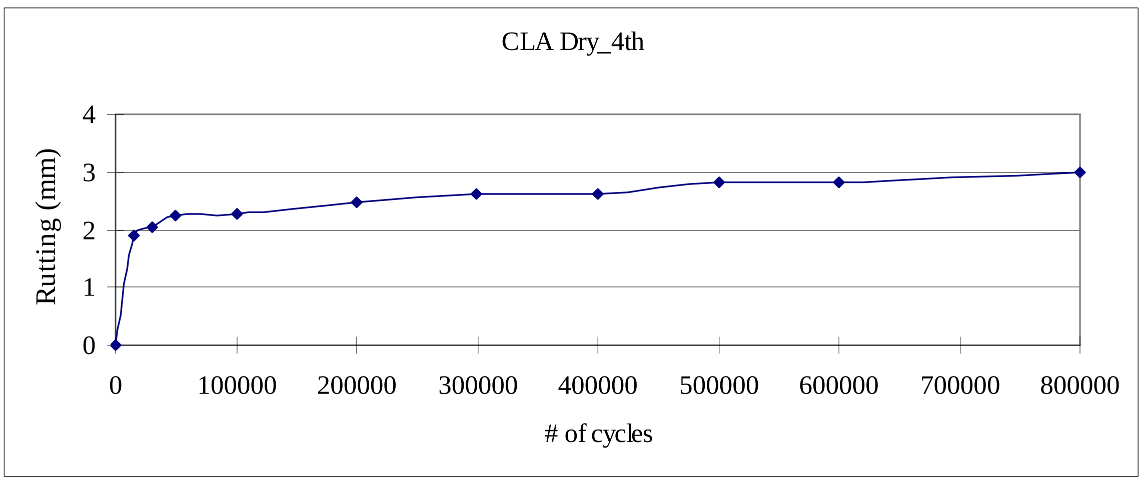

Figure B-8 Rutting versus number of cycles for CLA specimen in the 4th position under dry condition

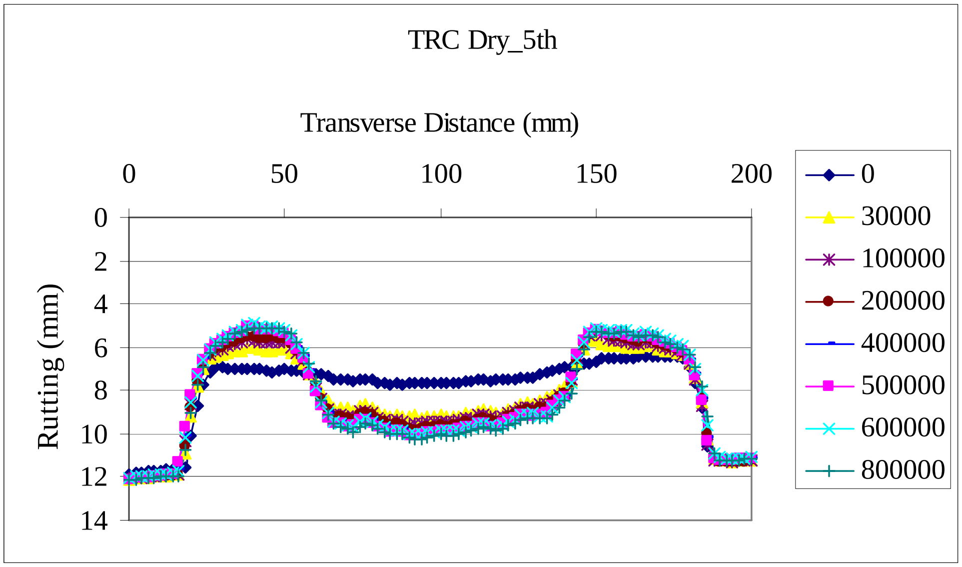

Figure B-9 Rutting versus transverse direction for TRC specimen in the 5th position under dry condition

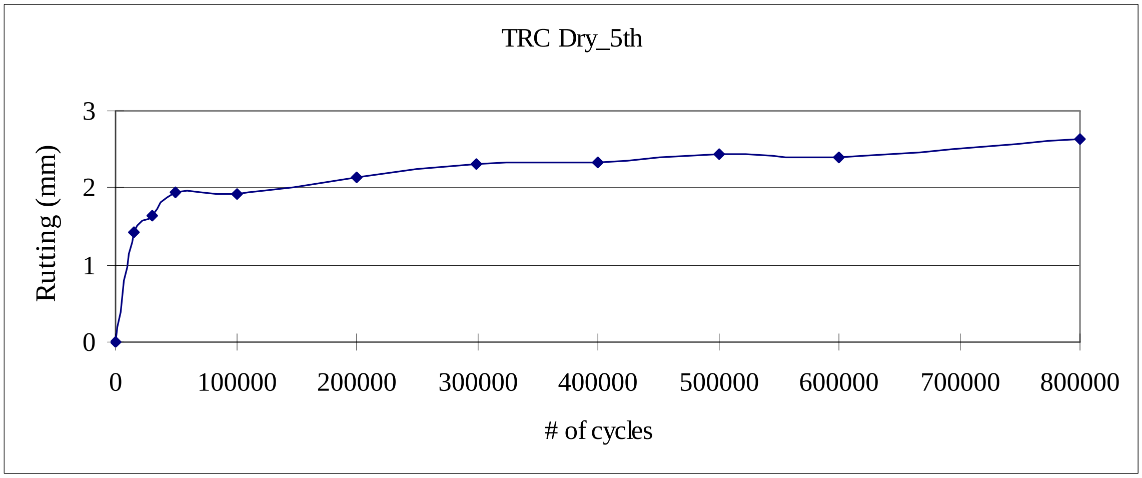

Figure B-10 Rutting versus number of cycles for TRC specimen in the 5th position under dry condition

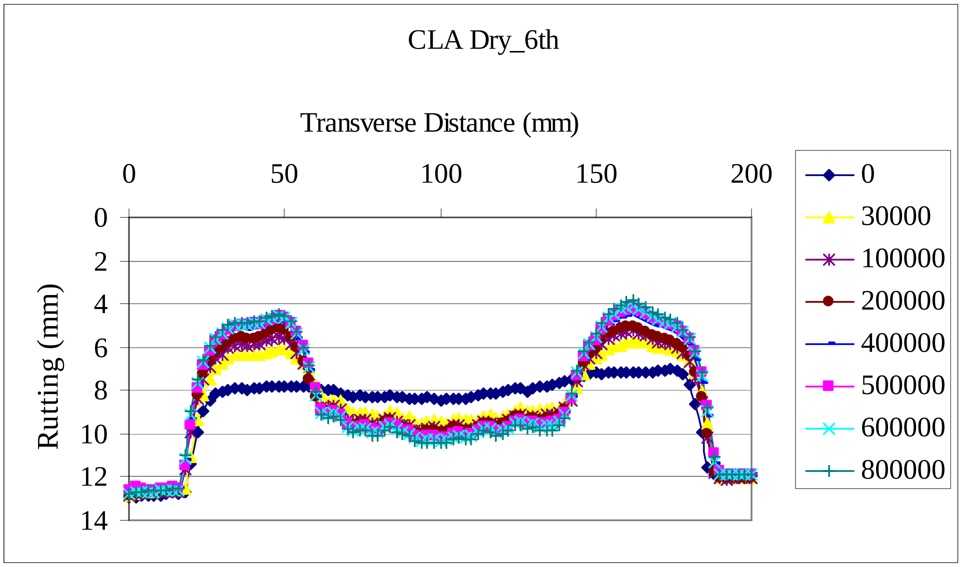

Figure B-11 Rutting versus transverse direction for CLA specimen in the 6th position under dry condition

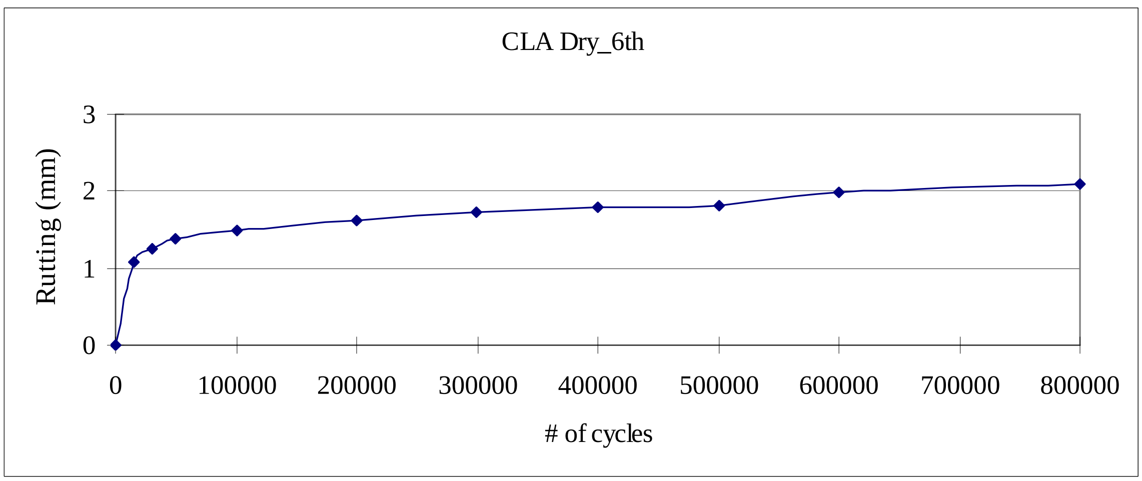

Figure B-12 Rutting versus number of cycles for CLA specimen in the 6th position under dry condition

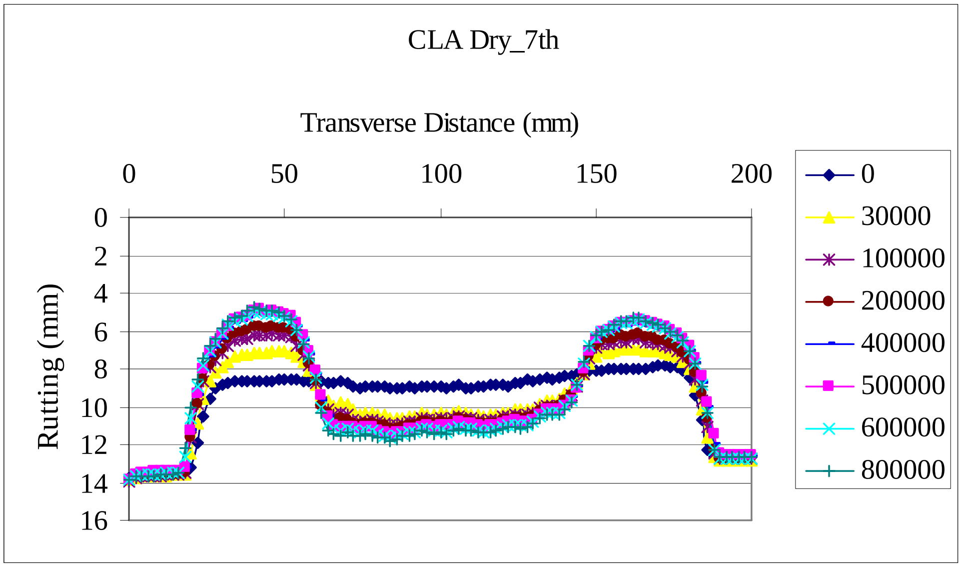

Figure B-13 Rutting versus transverse direction for CLA specimen in the 7th position under dry condition

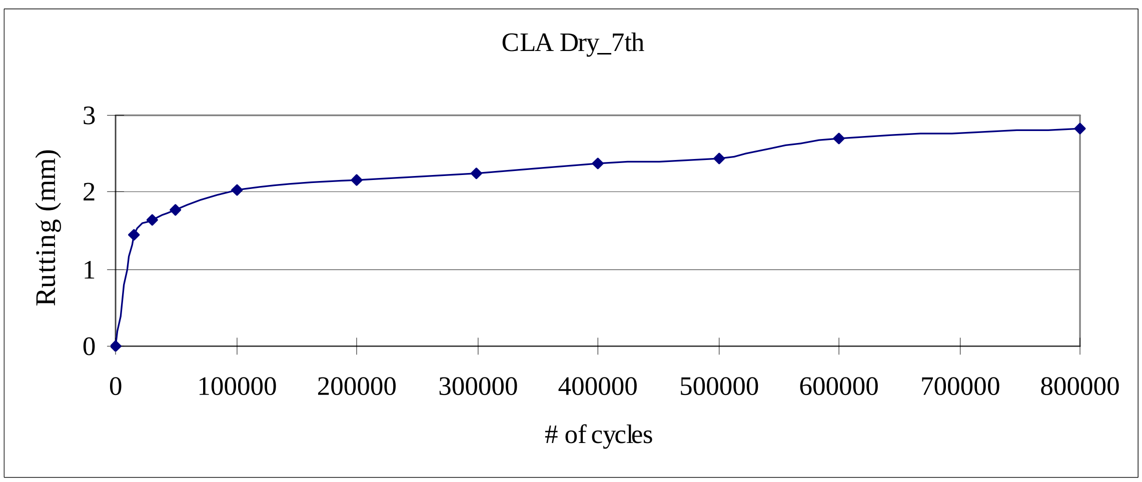

Figure B-14 Rutting versus number of cycles for CLA specimen in the 7th position under dry condition

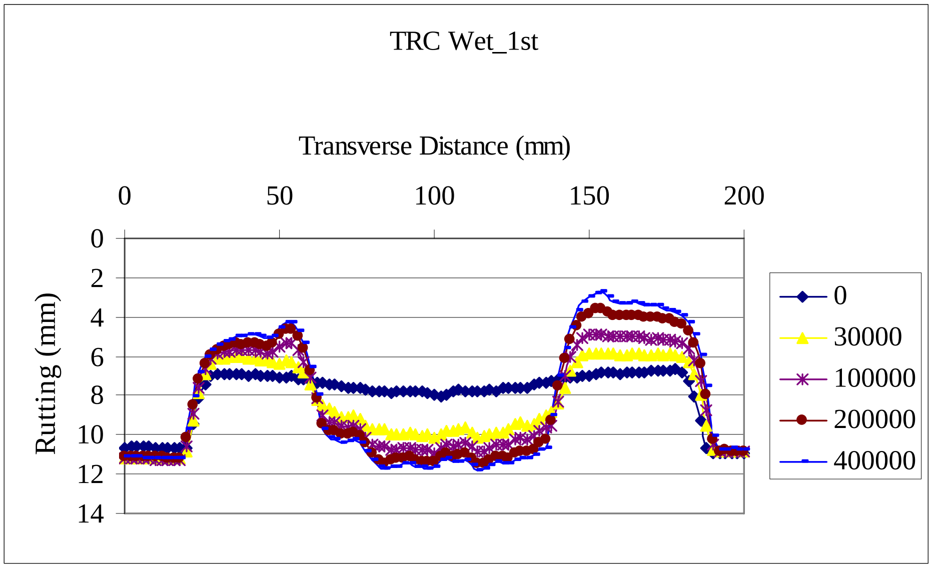

Figure B-15 Rutting versus transverse direction for TRC specimen in the 1st position under wet condition

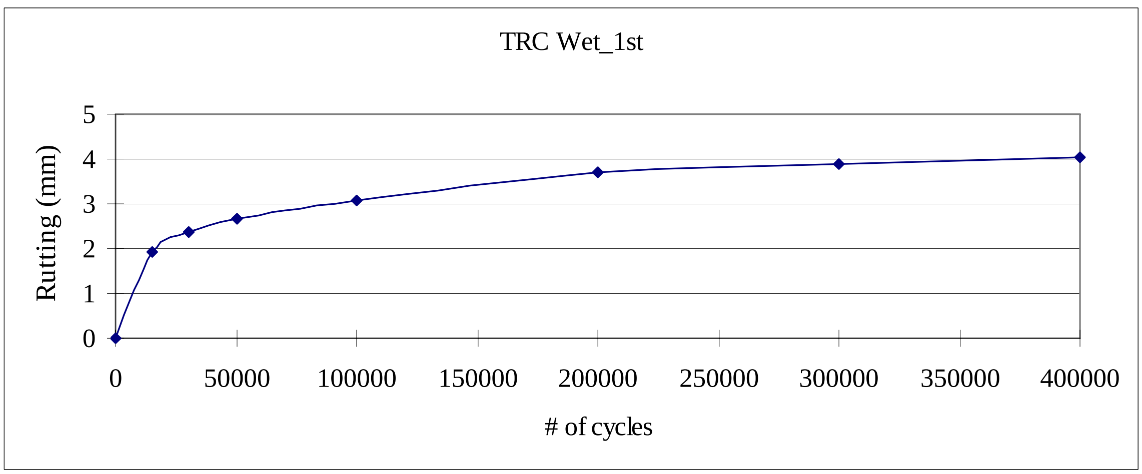

Figure B-16 Rutting versus number of cycles for TRC specimen in the 1st position under wet condition

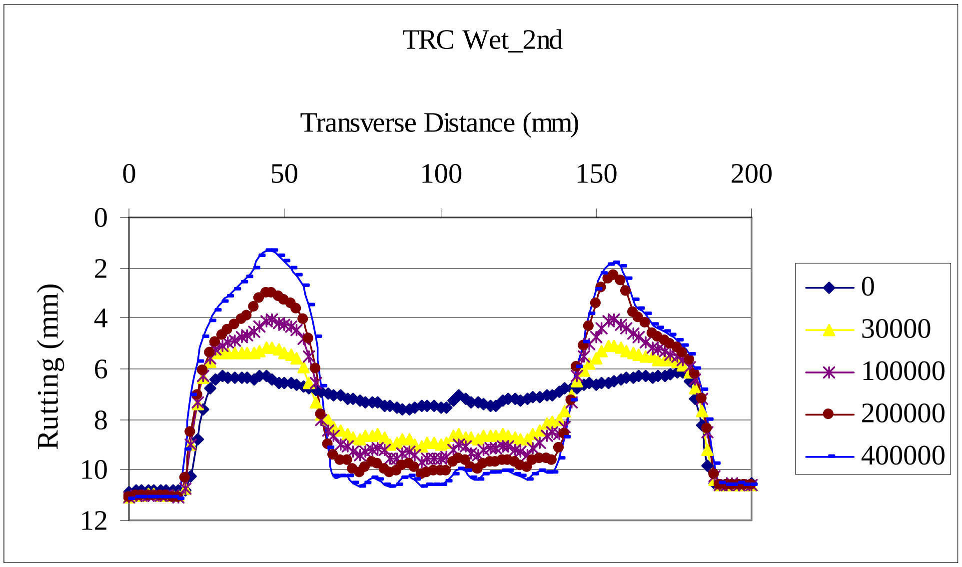

Figure B-17 Rutting versus transverse direction for TRC specimen in the 2nd position under wet condition

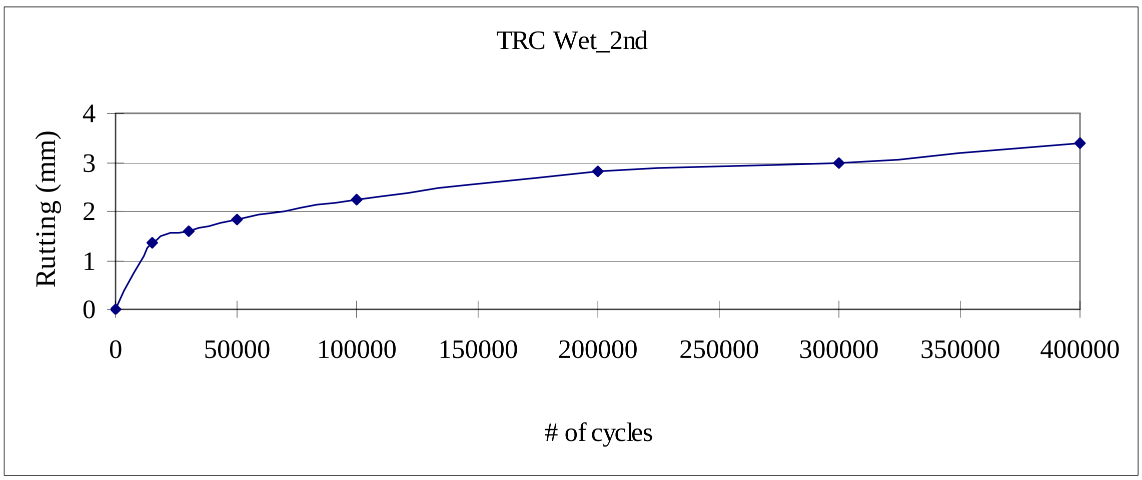

Figure B-18 Rutting versus number of cycles for TRC specimen in the 2nd position under wet condition

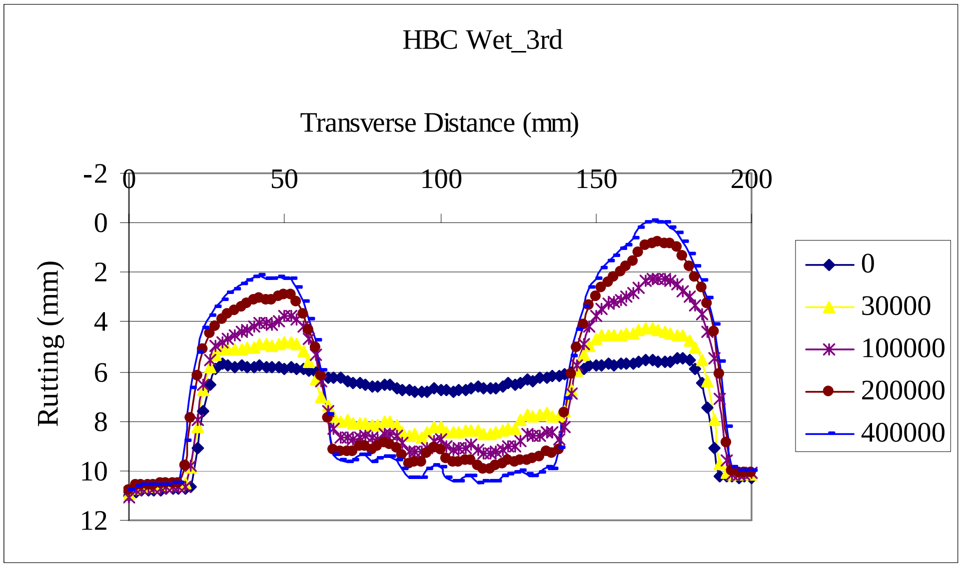

Figure B-19 Rutting versus transverse direction for HBC specimen in the 3rd position under wet condition

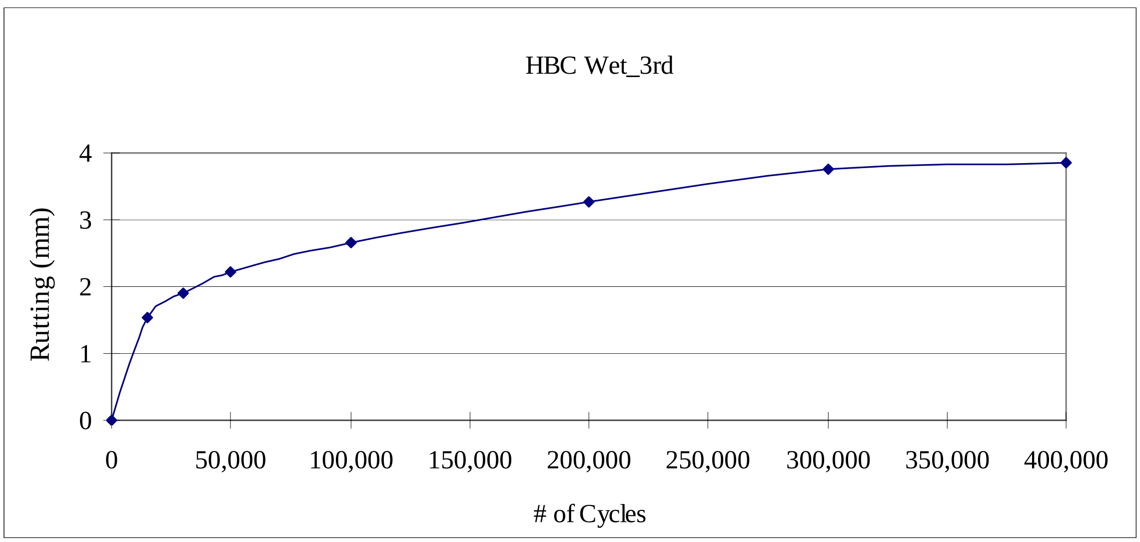

Figure B-20 Rutting versus number of cycles for HBC specimen in the 3rd position under wet condition

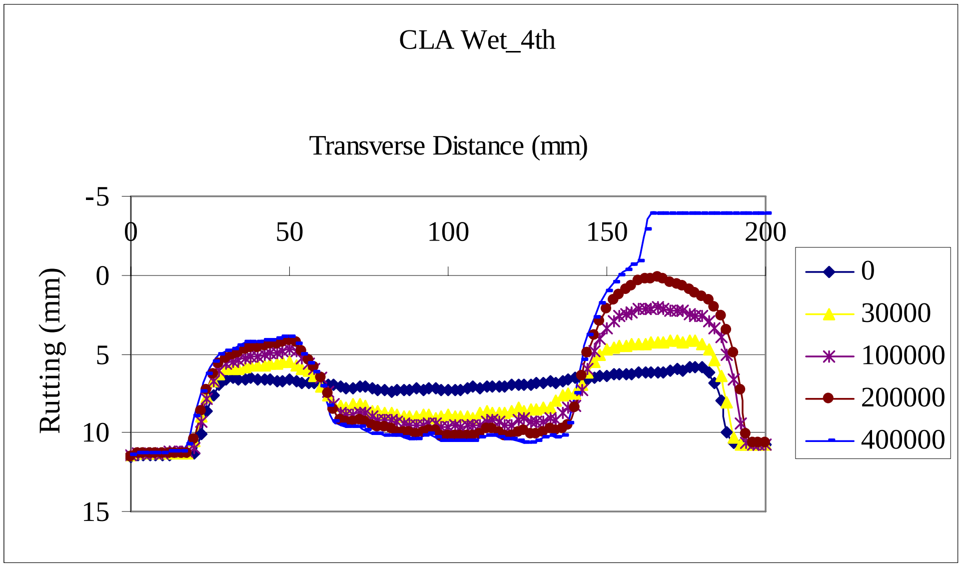

Figure B-21 Rutting versus transverse direction for CLA specimen in the 4th position under wet condition

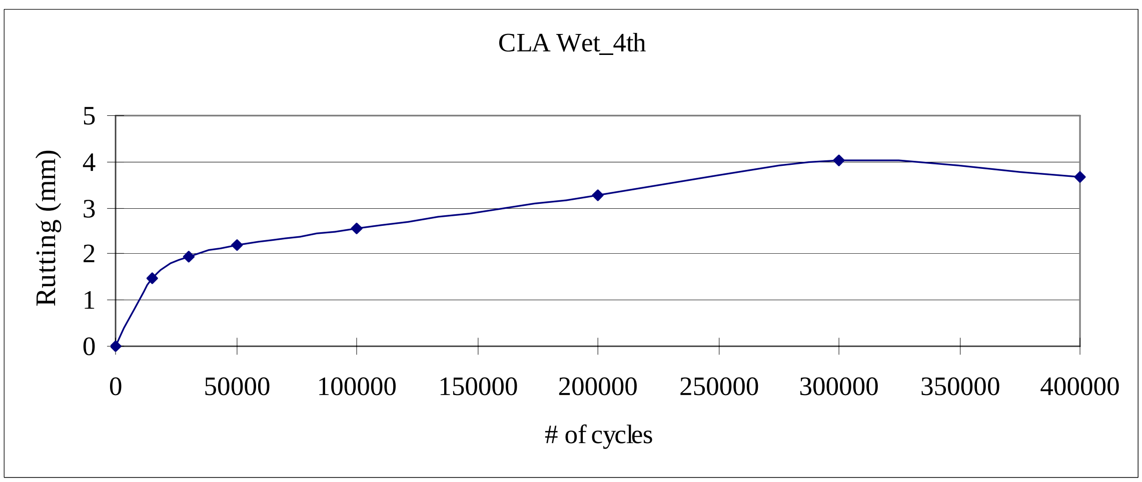

Figure B-22 Rutting versus number of cycles for CLA specimen in the 4th position under wet condition

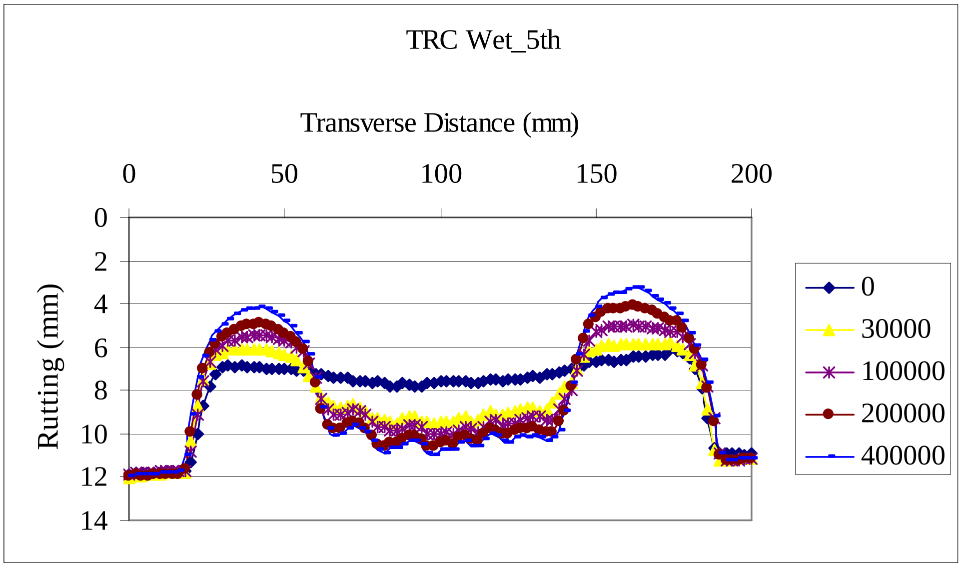

Figure B-23 Rutting versus transverse direction for TRC specimen in the 5th position under wet condition

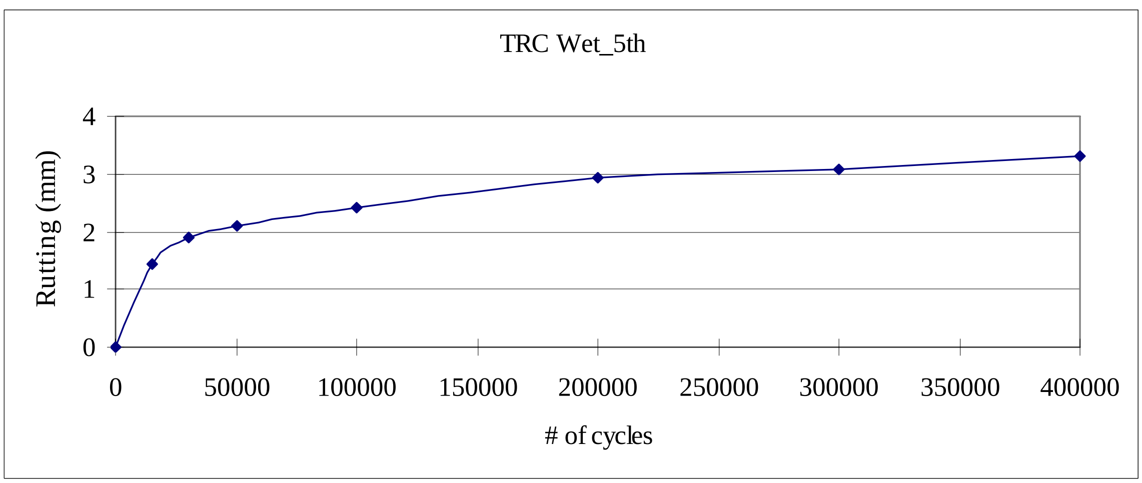

Figure B-24 Rutting versus number of cycles for TRC specimen in the 5th position under wet condition

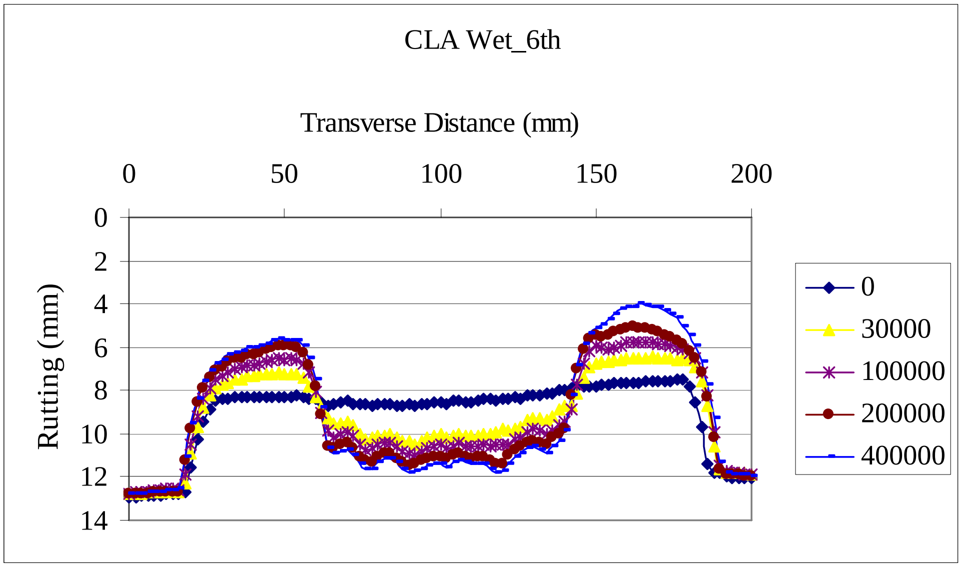

Figure B-25 Rutting versus transverse direction for CLA specimen in the 6th position under wet condition

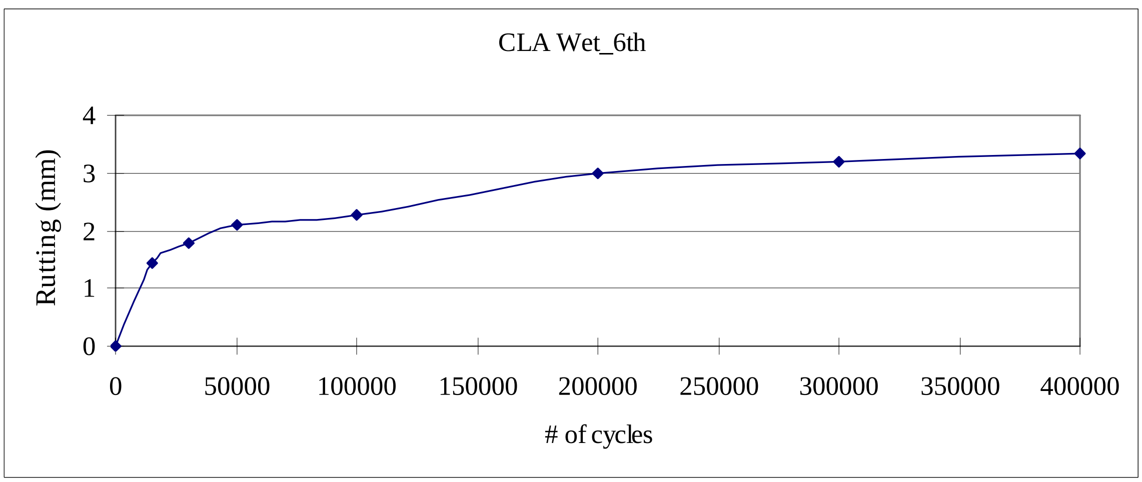

Figure B-26 Rutting versus number of cycles for CLA specimen in the 6th position under wet condition

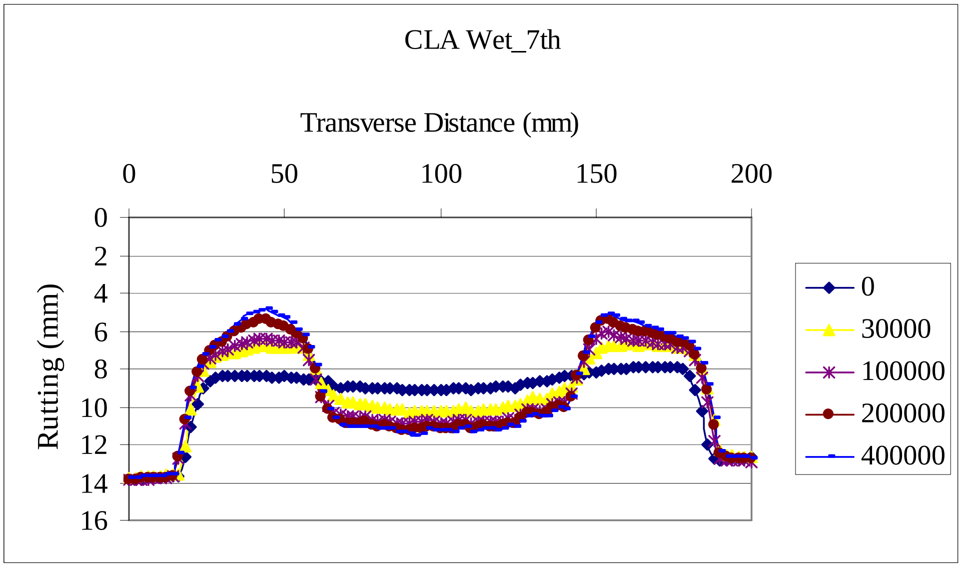

Figure B-27 Rutting versus transverse direction for CLA specimen in the 7th position under wet condition

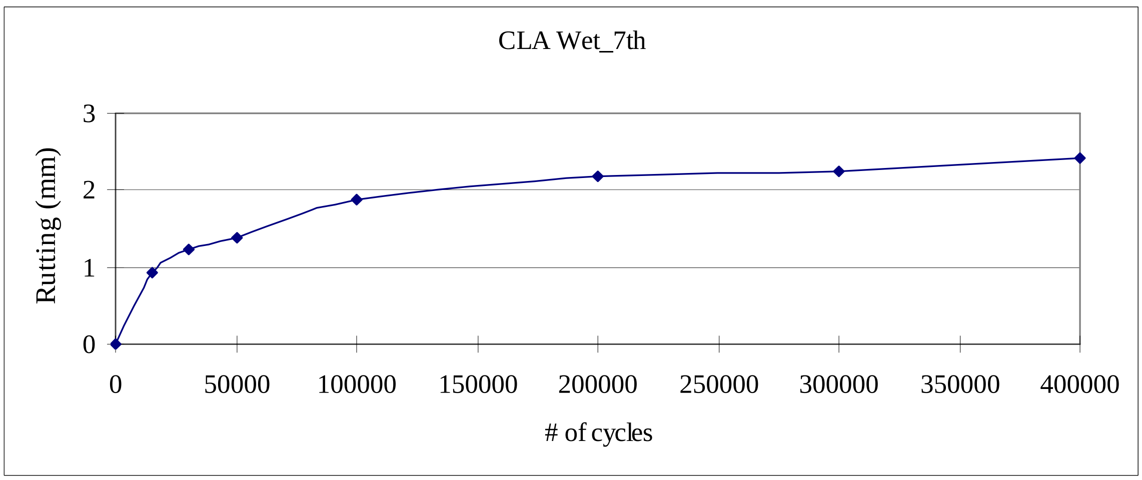

Figure B-28 Rutting versus number of cycles for CLA specimen in the 7th position under wet condition

APPENDIX

C

Detailed Results of Dynamic Modulus Tests

Table C-1 DM Test Results for TRC-C1

TRC1	Before 1st Conditioning				
Frequency	Level, %	microstrain	Load, kN	Modulus, MPa	Modulus, psi
10.00	2.00	55.62	1.82	4195.65	608369
5.00	1.30	55.81	1.38	3148.55	456539
2.00	1.00	64.00	1.12	2243.21	325266
1.00	0.70	65.33	0.84	1635.36	237127
TRC1	After 1st Conditioning				
Frequency	Level, %	microstrain	Load, kN	Modulus, MPa	Modulus, psi
10.00	1.50	64.76	1.38	2724.50	395053
5.00	0.80	58.10	0.90	1986.00	287970
2.00	0.50	58.67	0.67	1447.05	209822
1.00	0.30	49.14	0.49	1263.08	183147
TRC1	After 2nd Conditioning				
Frequency	Level, %	microstrain	Load, kN	Modulus, MPa	Modulus, psi
10.00	1.20	59.81	1.13	2402.83	348410
5.00	0.70	58.00	0.76	1703.57	247018
2.00	0.40	55.62	0.54	1246.91	180802
1.00	0.30	58.38	0.46	999.67	144953

Table C-2 DM Test Results of TRG2

TRC2	Before 1st Conditioning				
Frequency	Level, %	microstrain	Load, kN	Modulus, MPa	Modulus, psi
10.00	NA				
5.00					
2.00					
1.00					
TRC2	After 1st Conditioning				
Frequency	Level, %	microstrain	Load, kN	Modulus, MPa	Modulus, psi
10.00	1.50	53.05	1.42	3416.91	495452
5.00	0.80	42.86	0.91	2696.50	390993
2.00	0.50	37.71	0.66	2232.38	323695
1.00	0.30	38.10	0.48	1610.71	233554
TRC2	After 2nd Conditioning				
Frequency	Level, %	microstrain	Load, kN	Modulus, MPa	Modulus, psi
10.00	1.20	51.05	1.13	2843.58	412319
5.00	0.70	40.38	0.78	2482.31	359935
2.00	0.60	53.62	0.72	1713.19	248413
1.00	0.40	53.90	0.56	1318.02	191113

Table C-3 DM Test Results of TRG3

TRC3	Before 1st Conditioning				
Frequency	Level, %	microstrain	Load, kN	Modulus, MPa	Modulus, psi
10.00	1.65	48.00	1.61	4265.18	618451
5.00	1.08	47.52	1.14	3103.75	450043
2.00	0.83	55.90	0.94	2155.76	312585
1.00	0.55	42.57	0.70	2104.84	305202
TRC3	After 1st Conditioning				
Frequency	Level, %	microstrain	Load, kN	Modulus, MPa	Modulus, psi
10.00	1.30	55.14	1.20	2813.46	407951
5.00	1.00	67.29	1.04	1994.60	289217
2.00	0.70	45.81	0.82	2279.90	330585
1.00	0.40	49.29	0.56	1443.09	209248
TRC3	After 2nd Conditioning				
Frequency	Level, %	microstrain	Load, kN	Modulus, MPa	Modulus, psi
10.00	1.20	41.43	1.12	3495.09	506789
5.00	0.80	41.81	0.88	2729.82	395824
2.00	0.50	34.00	0.63	2368.46	343427
1.00	0.40	36.57	0.56	1959.83	284175

Table C-4 DM Test Results of CLA1

CLA1	Before 1st Conditioning				
Frequency	Level, %	microstrain	Load, kN	Modulus, MPa	Modulus, psi
10.00	1.50	57.52	1.45	3211.43	465658
5.00	1.00	51.14	1.08	2692.43	390402
2.00	0.70	52.38	0.83	2018.86	292735
1.00	0.50	45.52	0.65	1833.36	265837
CLA1	After 1st Conditioning				
Frequency	Level, %	microstrain	Load, kN	Modulus, MPa	Modulus, psi
10.00	1.10	48.00	1.03	2948.04	427466
5.00	0.80	55.43	0.88	2024.37	293533
2.00	0.40	44.19	0.54	1557.83	225885
1.00	0.30	43.24	0.47	1384.96	200819
CLA1	After 2nd Conditioning				
Frequency	Level, %	microstrain	Load, kN	Modulus, MPa	Modulus, psi
10.00	1.00	57.90	0.95	2108.64	305752
5.00	0.60	52.95	0.68	1648.31	239005
2.00	0.30	46.86	0.45	1210.24	175485
1.00	0.20	36.86	0.36	1263.15	183156

Table C-5 DM Test Results of CLA2

CLA2	Before 1st Conditioning				
Frequency	Level, %	microstrain	Load, kN	Modulus, MPa	Modulus, psi
10.00	1.30	45.52	1.24	3491.14	506216
5.00	0.90	43.05	0.96	2877.44	417229
2.00	0.70	48.38	0.83	2176.50	315593
1.00	0.50	43.24	0.66	1953.02	283188
CLA2	After 1st Conditioning				
Frequency	Level, %	microstrain	Load, kN	Modulus, MPa	Modulus, psi
10.00	1.10	48.48	1.04	2745.95	398162
5.00	0.80	44.10	0.88	2543.52	368811
2.00	0.50	38.95	0.64	2084.39	302237
1.00	0.35	35.14	0.51	1851.14	268415
CLA2	After 2nd Conditioning				
Frequency	Level, %	microstrain	Load, kN	Modulus, MPa	Modulus, psi
10.00	1.00	42.29	0.96	2916.42	422881
5.00	0.60	36.19	0.68	2390.97	346691
2.00	0.30	28.95	0.45	1980.44	287163
1.00	0.30	41.62	0.46	1400.44	203063

Table C-6 DM Test Results of CLA3

CLA3	Before 1st Conditioning				
Frequency	Level, %	microstrain	Load, kN	Modulus, MPa	Modulus, psi
10.00	1.30	37.24	1.23	4276.48	620089
5.00	1.10	50.76	1.16	2936.51	425794
2.00	0.70	44.86	0.83	2359.93	342190
1.00	0.50	38.86	0.65	2149.22	311637
CLA3	After 1st Conditioning				
Frequency	Level, %	microstrain	Load, kN	Modulus, MPa	Modulus, psi
10.00	1.10	41.14	1.04	3258.24	472444
5.00	0.90	48.10	0.97	2594.39	376187
2.00	0.50	39.62	0.64	2045.43	296587
1.00	0.30	29.71	0.46	1981.93	287380
CLA3	After 2nd Conditioning				
Frequency	Level, %	microstrain	Load, kN	Modulus, MPa	Modulus, psi
10.00	1.00	44.29	0.94	2701.43	391707
5.00	0.70	46.76	0.77	2142.70	310691
2.00	0.30	32.38	0.43	1695.39	245832
1.00	0.20	30.57	0.36	1488.73	215867