NCAT Report 02-12

NCAT TEST TRACK DESIGN, CONSTRUCTION, AND PERFORMANCE

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TABLE OF CONTENTS

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I. INTRODUCTION

A. Background

Empirical laboratory tests have been used for years to test HMA to determine the potential for various mixtures to perform well. As the amount of traffic has increased (higher volumes, higher loads, and increased tire pressures) the ability of these laboratory tests to evaluate potential performance has become more important. To keep up with these increased loads, the overall quality of the HMA has had to increase significantly to continue to provide satisfactory performance. Some of the deficiencies in material quality and construction procedures that were used in the past when traffic levels were lower have been corrected so that satisfactory performance has continued under these higher loads.

As the traffic has increased, better laboratory tests and material specifications have been developed to help ensure that high quality mixtures are produced. Superpave technology and SMA are two examples of improvements that have been made. Satisfactory performance under future loadings will require that our technology base continue to improve.

One of the problems in developing new tests is that it takes so many years to determine whether or not it truly does a good job of predicting performance or at least providing information that will allow subjective ranking of materials. Some tests that are not used to predict performance are still helpful in ensuring high quality mixes due to their ability to relatively rank the quality of the mixes. Better ways to evaluate the ability of laboratory tests to predict performance are needed. One way that has been used to evaluate new tests and materials is through the use of accelerated loading facilities. There are many types of facilities available including the ALF (Accelerated Loading Facility) which has been used by the FHWA and Louisiana, HVS (Heavy Vehicle Simulator) which has been used by Florida and California, Texas mobile load simulator used in Texas, and Purdue wheel tracking test facility. Many other facilities are available but not mentioned here. Recent test tracks have included WesTrack that was built in Reno, Nevada, and the MnRoad facility constructed in Minnesota. All of these accelerated load facilities have been widely used to help answer questions about pavement design and mixture types but much more work must be done.

It is difficult, in a short period of time, to develop the data needed to verify performance predictions based on new pavement design procedures or new performance tests. This can be done quickly in the laboratory but there is always the question of how well laboratory tests relate to performance. Collecting the data from in-place pavements takes many years since this requires the evaluation to go through a significant time period of traffic loading to collect the information needed. One way to decrease this time is to use accelerated loading facilities. Several procedures have been used to apply accelerated loading including mechanical devices that rapidly apply a given load to a test section(s) and test tracks with a number of test sections subjected to actual truck traffic.

Probably the most realistic way to test pavements under accelerated conditions is to apply actual trucks on a pavement test track. This procedure allows several test sections to be evaluated simultaneously. This approach also allows full scale test sections and actual trucks with typical loading to operate on these sections resulting in a loading situation that is very similar to that observed on the highways. It is generally felt that this test track approach is the most representative of what actually happens on the highways but it can be expensive to operate a number of trucks for an extended period of time. As a result of the advantages of a pavement test track the Alabama DOT in conjunction with NCAT decided that a test track was the best approach to provide practical answers to existing performance issues.

Unlike conventional efforts on public roadways, research at the NCAT Test Track is conducted on a closed-loop facility where axle loadings are monitored and environmental effects are similar for every mix. State DOT's typically have to wait 10 to 15 years to obtain results in full-scale field studies on public roadways where the traffic is not controlled. It is also often difficult to construct a number of sections at the same location so that equal traffic and subgrade conditions can be maintained among the different sections. There are also traffic control issues if the sections have to be inspected or repaired. So the test track approach is less disruptive to traffic, safer to the workers, and the amount of traffic applied to the test sections is better controlled.

The Test Track (referred to occasionally as the Track) is the result of industry and government committing to work together to improve the quality of flexible pavements. The facility is expected to clarify the relationship between methods and performance such that design and construction policy in the future can be objectively guided by life cycle costs.

The Alabama DOT funded the construction of the track at NCAT with anticipation that operation of the track would be a cooperative effort between several sponsors. Experimental sections on the 2.8 kilometer (1.7 miles) Test Track are cooperatively funded by external sponsors, most commonly state DOT's, with subsequent operation and research managed by NCAT. A total of 10,000,000 ESALs is applied over a two-year period of time, with subsequent pavement performance documented on a regular basis.

B. Objective

The primary objective of the test track was to provide an accelerated loading facility that could be used to rapidly test a large number of test sections simultaneously. This allows validation of laboratory tests and pavement design procedures under traffic similar to that which is observed on roadways. Based on the requirements of several sponsors several mini experiments were evaluated in the first cycle of testing. Some of the evaluations included: performance of fine graded vs. coarse graded mixes, effect of asphalt grade on performance, effect of aggregate type on performance, and performance of several mixture types including Superpave, SMA, and Open Graded Friction Courses. Other studies included the effect of grinding transverse joints on performance, effect of traffic on friction, permeability of various HMA mixtures, densification of HMA, and the effect of pavement smoothness on fuel consumption.

C. Scope of Work

This project involved working with 10 sponsors to develop an overall test plan to evaluate the effect of several mixture types and properties on performance. Aggregates were hauled in from the various states to construct the test sections. Over 60 different stockpiles were selected to be used to construct the test sections. Generally, each sponsor provided funds to pay the expenses of constructing, testing, and analyzing two sections on the track. The track consisted of 26 sections in the tangents and another 20 sections in the curves. Initially, only the tangents were going to be evaluated but it was decided that the curves should not be completely wasted, hence test sections were also constructed in the curves. The pavement was designed with sufficient thickness to ensure that no structural damage would occur during testing and hence failures should be limited to the surface or near surface layers of the HMA.

Each sponsor was allowed to establish the test program to be used for their sections. In some cases, sponsors cooperated in developing a larger study by pooling their sections into a larger overall program. After the test sections were built a total of 10,000,000 ESALs were applied over a 2 year period. The ESALs were applied with 4 fully loaded trucks with 3 trailers per tractor. Each tractor pulled a load of approximately 152,000 pounds, 20,000 for each of 7 loaded axles and approximately 12,000 pounds for the front steer axle.

The condition of each section was monitored weekly to evaluate rutting, cracking, and other surface related problems. On a monthly basis testing was conducted to evaluate friction, roughness, falling weight deflectometer measurements, and densification. Instrumentation was placed in the pavement structure during construction to determine moisture content in the improved subgrade material and temperature at 4 elevations in the pavement structure. This information was collected on a continuous basis throughout the life of the project.

At the end of the project the measured performance was correlated to various mixture and material properties to help identify those properties that could be used to ensure good performance.

II. SPONSORS

One of the advantages of a test track is that it allows several sections to be constructed and trafficked at one time so that a direct comparison can be made between the sections. Because of the higher cost of constructing and testing several sections simultaneously, several sponsors were needed to help finance the operation of the facility. The sponsors of the first cycle at the track included: Alabama DOT, Florida DOT, Georgia DOT, Indiana DOT, Mississippi DOT, North Carolina DOT, Oklahoma DOT, South Carolina DOT, Tennessee DOT, and the Federal Highway Administration (FHWA). A lot of support was provided from the HMA industry including APAC, Inc., ASTEC Industries, Caterpillar, Inc. Compaction America, Vulcan Materials, Ergon, Inc. and Koch Materials.

III. EXPERIMENTAL DESIGN

An oversight committee was formed at the beginning of this study in which sponsors were encouraged to work together as much as they could so that an overall test plan for the facility could be developed. Most sponsors chose to ship in their own local aggregates while using common asphalt binders that were used for most of the test sections. Table 1 is included herein to provide an overall summary of the various test sections.

One of the primary purposes of the first cycle of tests was to determine the ability of a number of laboratory tests to predict the permanent deformation of various HMA mixtures. There was no specific design established to do this since each sponsor was allowed to use any mix that they desired. However, this approach did provide a wide range of mixture types and properties and hence provided the opportunity to establish any relationship that may exist between performance and laboratory tests.

There were some sponsors that were interested in comparing fine graded vs. coarse graded mixes. These test sections offered the opportunity to determine the effect of aggregate grading on performance.

Several aggregates were used on the track including: limestone, granite, marine limestone, gravel, and slag. Reclaimed Asphalt Pavement (RAP) was also used in a few sections. These test sections provided some opportunity to evaluate the effect of aggregate type on performance.

There were several direct comparisons of mixtures containing PG76-22 and PG 67-22 while all other mix properties were held constant. This allowed a direct comparison of the performance of mixes containing the two grades of AC.

On some occasions an additional 0.5% asphalt cement was added to mixtures to determine the effect of extra binder. This was done for the modified as well as for the unmodified binders.

So, while there was no overall experimental design there were several small experimental designs that allowed some answers to be obtained that could be used locally. The information from the smaller efforts was then combined to determine overall findings. From a design standpoint, it would have been better to have an overall design with each section fitting into this design. However, the various sponsors had issues that they wanted to evaluate leading to several mini designs. This approach did not create any significant problem and was certainly desirable from the viewpoint of the individual states.

Table 1. Overview of Mix Types Evaluated

Track Quad	Section Num	Aggregate Blend Type	Design Method	Design NMA	Grad Type	Binder Grade	Binder Modifier	Lift Type	Design Thick	Survey Thick
E	**2**	Granite	Super	12.5	BRZ	67-22	NA	Dual	4.0	4.2
E	**3**	Granite	Super	12.5	BRZ	76-22	SBR	Dual	4.0	4.1
E	**4**	Granite	Super	12.5	BRZ	76-22	SBS	Dual	4.0	4.1
E	**5**	Granite	Super	12.5	TRZ	76-22	SBS	Dual	4.0	4.2
E	**6**	Granite	Super	12.5	TRZ	67-22	NA	Dual	4.0	4.2
E	**7**	Granite	Super	12.5	TRZ	76-22	SBR	Dual	4.0	4.2
E	**8**	Granite	Super	12.5	ARZ	67-22	NA	Dual	4.0	4.2
E	**9**	Granite	Super	12.5	ARZ	76-22	SBS	Dual	4.0	4.1
E	**10**	Granite	Super	12.5	ARZ	76-22	SBR	Dual	4.0	4.4
N	**1**	Slag/Lms	Super	12.5	ARZ	76-22	SBS	Dual	4.0	3.9
N	**2**	Slag/Lms	Super	12.5	ARZ	76-22+	SBS	Dual	4.0	4.3
N	**3**	Slag/Lms	Super	12.5	ARZ	67-22+	NA	Dual	4.0	4.2
N	**4**	Slag/Lms	Super	12.5	ARZ	67-22	NA	Dual	4.0	4.2
N	**5**	Slag/Lms	Super	12.5	BRZ	67-22+	NA	Dual	4.0	4.4
N	**6**	Slag/Lms	Super	12.5	BRZ	67-22	NA	Dual	4.0	4.1
N	**7**	Slag/Lms	Super	12.5	BRZ	76-22+	SBR	Dual	4.0	3.9
N	**8**	Slag/Lms	Super	12.5	BRZ	76-22	SBR	Dual	4.0	3.9
N	**9**	Slag/Lms	Super	12.5	BRZ	76-22	SBS	Dual	4.0	3.9
N	**10**	Slag/Lms	Super	12.5	BRZ	76-22+	SBS	Dual	4.0	4.2
N	**11**	Granite	Super	19.0	BRZ	67-22	NA	Lower	2.5	NA
		Granite	Super	12.5	TRZ	76-22	SBS	Upper	1.5	4.1
N	**12**	Granite	Super	19.0	BRZ	67-22	NA	Lower	2.5	NA
		Granite	SMA	12.5	SMA	76-22	SBS	Upper	1.5	3.9
N	**13**	Gravel	Super	19.0	BRZ	76-22	SBS	Lower	2.5	NA
		Gravel	SMA	12.5	SMA	76-22	SBS	Upper	1.5	4.0
W	**1**	Granite	SMA	12.5	SMA	76-22	SBR	Dual	4.0	3.9
W	**2**	Slag/Lms	SMA	12.5	SMA	76-22	SBR	Dual	4.0	4.0
W	**3**	Granite	Super	12.5	BRZ	76-22	SBR	Lower	3.3	NA
		Slag/Lms	OGFC	12.5	OGFC	76-22	SBR	Upper	0.7	4.0
W	**4**	Limestone	SMA	12.5	SMA	76-22	SBR	Lower	3.3	NA
		Granite	OGFC	12.5	OGFC	76-22	SBR	Upper	0.7	4.1
W	**5**	Limestone	SMA	12.5	SMA	76-22	SBS	Lower	3.3	NA
		Granite	OGFC	12.5	OGFC	76-22	SBS	Upper	0.7	4.3
W	**6**	Slag/Lms	Super	12.5	TRZ	67-22	NA	Dual	4.0	4.1
W	**7**	Limestone	SMA	12.5	SMA	76-22	SBR	Dual	4.0	4.2
W	**8**	Sandstn/Slg/Lms	SMA	12.5	SMA	76-22	SBR	Dual	4.0	4.0
W	**9**	Gravel	Super	12.5	BRZ	67-22	NA	Dual	4.0	4.0
W	**10**	Gravel	Super	12.5	BRZ	76-22	SBR	Dual	4.0	3.9
S	**1**	Granite	Super	19.0	BRZ	76-22	SBS	Lower	2.5	NA
		Granite	Super	12.5	BRZ	76-22	SBS	Upper	1.5	3.9
S	**2**	Gravel	Super	19.0	BRZ	76-22	SBS	Lower	2.5	NA
		Gravel	Super	9.5	BRZ	76-22	SBS	Upper	1.5	3.9
S	**3**	Limestone	Super	19.0	BRZ	76-22	SBS	Lower	2.5	NA
		Lms/Gravel	Super	9.5	BRZ	76-22	SBS	Upper	1.5	4.0
S	**4**	Lms/RAP	Super	19.0	ARZ	76-22	SBS	Lower	2.5	NA
		Limestone	Super	12.5	ARZ	76-22	SBS	Upper	1.5	4.0
S	**5**	Lms/Grv/RAP	Super	19.0	BRZ	76-22	SBS	Lower	2.5	NA
		Gravel	Super	12.5	TRZ	76-22	SBS	Upper	1.5	4.1
S	**6**	Lms/RAP	Super	12.5	ARZ	67-22	NA	Dual	4.0	4.1
S	**7**	Lms/RAP	Super	12.5	BRZ	67-22	NA	Dual	4.0	4.0
S	**8**	Marble-Schist	Super	19.0	BRZ	67-22	NA	Lower	2.1	NA
		Marble-Schist	Super	12.5	BRZ	76-22	SBS	Upper	1.5	3.8
S	**9**	Granite	Super	12.5	BRZ	67-22	NA	Dual	3.0	3.0
S	**10**	Granite	Super	12.5	ARZ	67-22	NA	Dual	3.0	3.1
S	**11**	Marble-Schist	Super	19.0	BRZ	67-22	NA	Lower	2.1	NA
		Marble-Schist	Super	9.5	BRZ	76-22	SBS	Upper	1.5	3.6
S	**12**	Limestone	Hveem	12.5	TRZ	70-28	SB	Dual	4.0	3.8
S	**13**	Granite	Super	12.5	ARZ	70-28	SB	Dual	4.0	4.0
E	**1**	Gravel	Super	12.5	ARZ	67-22	NA	Dual	4.0	4.1

Notes: - Mixes are listed chronologically in order of completion dates.
- "dual" lift type indicates that the upper and lower lifts were constructed with the same mix.
- ARZ, TRZ, and BRZ refer to gradations intended to pass above, through, and below the restricted zone, respectively
- SMA and OGFC refer to stone matrix asphalt and open-graded friction course mixes, respectively.

IV. MATERIAL AND MIXTURE PROPERTIES

The materials used for this project were selected by the various sponsors. Most states used their local aggregate for testing. Common sources of binder were available for use and were utilized by most of the states.

The constructed material and mixture properties for the tangents and curves are shown in Table 2 for the surface course only.

V. CONSTRUCTION

A. Pavements

APAC, Couch Division was selected to build the Track through a competitively bid contract administered by the Alabama Department of Transportation (ALDOT). As a condition of the work, the contractor was required to supply an onsite plant, a material transfer device (MTD), a rubber-tired roller as well as other conventional rollers, and a host of other equipment (each meeting a particular specification requirement). All of this equipment was useful in producing mixes that met the specification requirements and that was satisfactory to engineers from the various sponsors. Aggregate stockpiles and asphalt binders were hauled in from eight different states in order for the research to adequately reflect the local interests of the sponsors. Field and laboratory technicians representing the sponsors were on board during construction of their sections to review NCAT-generated results and provide guidance on section quality and final acceptance.

Table 2. As Constructed Mixture Properties (surface course)

Section	E1	E2	E3	E4	E5	E6	E7	E8	E9	E10
Gradation Type	ARZ	BRZ	BRZ	BRZ	TRZ	TRZ	TRZ	ARZ	ARZ	ARZ
Aggregate Type	Quartzite	Granite	Granite	Granite	Granite	Granite	Granite	Granite	Granite	Granite
1"	100	100	100	100	100	100	100	100	100	100
3/4"	100	100	100	100	100	100	100	100	100	100
1/2"	99	96	94	95	98	96	97	98	97	97
3/8"	92	74	73	75	83	81	83	86	85	87
No. 4	73	41	41	42	54	52	53	66	64	67
No. 8	54	29	29	29	40	37	38	51	49	51
No. 16	38	22	23	23	30	28	29	38	36	38
No. 30	25	18	18	18	24	22	22	28	27	29
No. 50	14	12	12	13	16	15	16	18	18	19
No. 100	9	7	7	8	9	8	9	10	10	10
No. 200	7.4	4.1	4.2	4.6	5.1	4.3	5.2	5.2	5.2	5.6
Average QC Lab Air Voids	3.3%	2.8%	4.0%	3.8%	3.7%	3.9%	3.6%	4.2%	4.4%	3.5%
Compactive Effort*	G100	G100	G100	G100	G100	G100	G100	G100	G100	G100
In-Place Air Voids	6.0%	5.3%	6.5%	6.2%	7.3%	7.1%	6.8%	7.3%	7.1%	7.0%
Asphalt Content	5.3%	4.7%	4.8%	4.7%	5.1%	5.0%	4.8%	5.6%	5.4%	5.8%
PG Grade	67-22	67-22	76-22	76-22	76-22	67-22	76-22	67-22	76-22	76-22
Modifier Type	NA	NA	SBR	SBS	SBS	NA	SBR	NA	SBS	SBR

*A number following the G prefix indicates the number of gyrations in the gyratory compactor.
*A number following the M prefix indicates the number of blows by Marshall Hammer.

Table 2 (continued) As Constructed Mixture Properties (surface course)

Section	N1	N2	N3	N4	N5	N6	N7	N8	N9	N10	N11	N12	N13
Gradation Type	ARZ	ARZ	ARZ	ARZ	BRZ	BRZ	BRZ	BRZ	BRZ	BRZ	TRZ	SMA	SMA
Aggregate Type	Lms/Slag	Lms/Slag	Lms/Slag	Lms/Slag	Lms/Slag	Lms/Slag	Lms/Slag	Lms/Slag	Lms/Slag	Lms/Slag	Granite	Granite	Gravel
1"	100	100	100	100	100	100	100	100	100	100	100	100	100
3/4"	100	100	100	100	100	100	100	100	100	100	100	100	100
1/2"	100	99	99	99	99	99	98	99	99	98	97	96	99
3/8"	92	90	91	91	84	85	83	85	87	84	80	73	74
No. 4	69	66	68	68	52	54	52	55	57	51	52	32	30
No. 8	52	50	51	52	38	37	36	37	40	34	37	23	25
No. 16	33	33	33	35	26	25	24	24	26	23	30	21	23
No. 30	22	22	22	23	18	17	17	17	19	17	24	19	21
No. 50	15	16	15	15	14	13	13	13	14	13	18	17	17
No. 100	10	11	10	9	11	10	10	10	11	10	11	14	13
No. 200	6.7	7.6	6.5	6.0	8.3	8.2	7.8	7.5	8.8	7.7	7.2	11.8	11.5
Average QC Lab Air Voids	2.5%	2.2%	3.2%	4.3%	3.0%	3.3%	2.1%	4.0%	3.2%	3.5%	3.4%	2.7%	4.0%
Compactive Effort*	G100	G100	G100	G100	G100	G100	G100	G100	G100	G100	G100	M50	M50
In-Place Air Voids	4.9%	5.3%	5.9%	6.6%	6.2%	5.6%	6.1%	5.3%	5.5%	5.3%	6.9%	5.4%	8.0%
Asphalt Content	7.4%	7.8%	7.6%	6.8%	6.9%	6.8%	6.9%	6.6%	6.7%	6.8%	4.3%	6.2%	6.8%
PG Grade	76-22	76-22	67-22	67-22	67-22	67-22	76-22	76-22	76-22	76-22	76-22	76-22	76-22
Modifier Type	SBS	SBS	NA	NA	NA	NA	SBR	SBR	SBS	SBS	SBS	SBS	SBS

*A number following the G prefix indicates the number of gyrations in the gyratory compactor.
*A number following the M prefix indicates the number of blows by Marshall Hammer.

Table 2 (continued) As Constructed Mixture Properties (surface course)

Section	**W1**	**W2**	**W3**	**W4**	**W5**	**W6**	**W7**	**W8**	**W9**	**W10**
Gradation Type	SMA	SMA	OGFC	OGFC	OGFC	TRZ	OGFC	SMA	BRZ	BRZ
Aggregate Type	Granite	Lms/Slag	Lms/Slag	Granite	Granite	Lms/Slag	Granite	Sandstone	Qtz gravel	Qtz gravel
1"	100	100	100	100	100	100	100	100	100	100
3/4"	100	100	100	100	100	100	100	100	100	100
1/2"	95	98	98	95	95	99	95	99	96	96
3/8"	68	77	68	66	67	89	74	80	80	81
No. 4	28	35	19	23	22	65	32	33	51	51
No. 8	20	24	13	14	15	45	23	25	34	33
No. 16	18	17	11	13	12	28	18	22	22	22
No. 30	16	15	10	12	11	18	15	20	16	16
No. 50	14	13	9	11	11	13	12	18	12	12
No. 100	12	12	8	10	10	10	9	15	9	9
No. 200	9.7	10.7	6.8	8.6	8.5	7.8	5.9	12.9	6.7	6.5
Average QC Lab Air Voids	3.5%	3.8%	NA	NA	NA	2.7%	NA	3.5%	3.4%	4.0%
Compactive Effort*	M50	M50	NA	NA	NA	G100	NA	M50	G100	G100
In-Place Air Voids	5.0%	5.7%	NA	NA	NA	7.9%	NA	5.5%	6.4%	6.7%
Asphalt Content	6.1%	8.0%	7.6%	6.1%	6.2%	6.8%	4.8%	7.5%	5.0%	5.0%
PG Grade	76-22	76-22	76-22	76-22	76-22	67-22	76-22	76-22	67-22	76-22
Modifier Type	SBR	SBR	SBR	SBR	SBS	NA	SB	SBR	NA	SBR

*A number following the G prefix indicates the number of gyrations in the gyratory compactor.
*A number following the M prefix indicates the number of blows by Marshall Hammer.

Table 2 (continued) As Constructed Mixture Properties (surface course)

Section	**S1**	**S2**	**S3**	**S4**	**S5**	**S6**	**S7**	**S8**	**S9**	**S10**	**S11**	**S12**	**S13**
Gradation Type	BRZ	BRZ	BRZ	ARZ	TRZ	ARZ	BRZ	BRZ	BRZ	ARZ	BRZ	TRZ	ARZ
Aggregate Type	Granite	Gravel	Lms/gravel	Limestone	Gravel	Lms/RAP	Lms/RAP	Marble Schist	Granite	Granite	Marble Schist	Limestone	Granite
1"	100	100	100	100	100	100	100	100	100	100	100	100	100
3/4"	100	100	100	100	100	100	100	100	100	100	100	100	100
1/2"	95	100	100	98	95	95	96	100	93	95	100	97	93
3/8"	86	96	100	88	82	87	88	93	82	88	92	82	80
No. 4	54	67	70	63	61	74	71	58	53	69	62	63	68
No. 8	36	41	43	46	45	53	34	38	36	52	47	46	50
No. 16	28	29	29	33	33	41	25	25	27	38	30	32	37
No. 30	21	22	21	23	22	33	20	19	20	27	22	23	27
No. 50	15	15	15	13	10	24	16	15	14	19	17	16	19
No. 100	9	10	11	9	7	12	10	12	9	11	13	10	11
No. 200	5.5	8.4	8.9	7.8	5.0	5.9	6.2	7.8	5.7	6.6	7.5	7.0	6.6
Average QC Lab Air Voids	3.0%	4.7%	3.5%	2.2%	3.4%	4.5%	3.3%	2.7%	3.6%	3.2%	3.1%	3.8%	4.8%
Compactive Effort*	G100	G100	G100	G125	G125	G100	G100	G100	G100	G100	G100	NA	G100
In-Place Air Voids	5.2%	6.2%	7.3%	5.7%	5.1%	7.1%	6.8%	8.2%	6.6%	6.3%	6.8%	6.1%	6.6%
Asphalt Content	5.0%	6.0%	5.6%	5.3%	5.6%	6.2%	6.6%	4.2%	4.7%	5.2%	3.9%	4.5%	5.3%
PG Grade	76-22	76-22	76-22	76-22	76-22	67-22	67-22	76-22	67-22	67-22	76-22	70-28	70-28
Modifier Type	SBS	SBS	SBS	SBS	SBS	NA	NA	SBS	NA	NA	SBS	SB	SB

*A number following the G prefix indicates the number of gyrations in the gyratory compactor.
*A number following the M prefix indicates the number of blows by Marshall Hammer.

Trial mix was originally run through the plant with blend percentages set on the job mix formula. This "waste" material was used for paving improvements at the plant site, placed on the formerly unpaved county access road to the facility, or placed in a waste stockpile and made available for local maintenance activities. In most cases it was necessary to make adjustments to the laboratory job mix formula before placement operations were allowed to begin. Subsequent paving on the Track was allowed to commence only when section sponsors were satisfied that the quality of the mix would meet their research expectations.

Beginning with the second section in the East curve, paving operations proceeded around the oval in a counterclockwise manner (Figure 1). Enough mix was produced with each plant production run to facilitate placement of both the inside and outside lanes of the lift under construction. Inside lanes were paved first so that satisfactory roller patterns could be identified and utilized in the more critical outside (research) lane. It was found early on that with the inherently tight working area and excessive amount of equipment within the limits of the 61 meter sections (200 ft.), it would not be possible to pave lower and upper lifts of a section within the same workday without damaging the fresh mat; consequently, lower lifts were paved at least one day ahead of upper lifts to enhance overall construction quality.

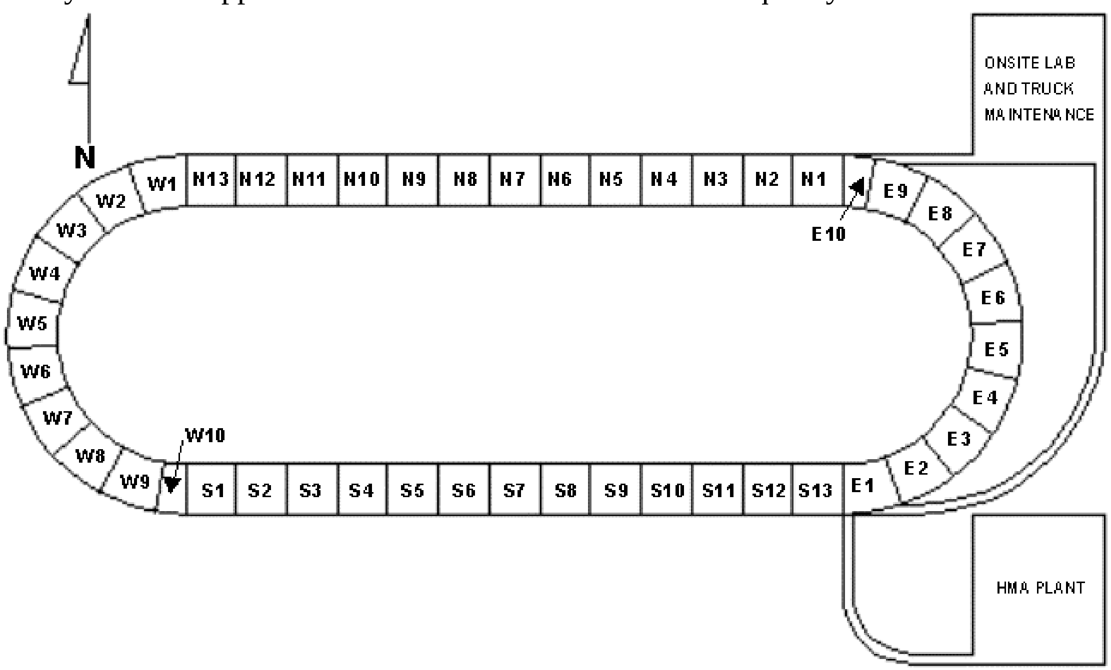

Figure 1. Layout of Test Track

The first lower lift was placed in the second section of the east curve on March 21, 2000. Work proceeded in a counter-clockwise manner around the Track through the spring and into summer. The east curve was completed, followed by the north tangent, the west curve, and finally the south tangent. The last surface course was placed on the first section of the east curve on July 14, 2000.

Laboratory job-mix formulas were used as a starting point when each mix was trial run through the plant for the first time. Stockpile moisture contents were measured daily on any aggregates that were scheduled for production to minimize the effect on plant operations and resulting final mix proportions. A portable double drum plant (presented as Figure 2) was temporarily located onsite to produce mix exclusively for Track construction with minimal haul times.

Figure 2. Onsite Double Drum Asphalt Plant Used to Produce All Track Mix

A sufficient quantity of mix was wasted on either end of each production run so that a meaningful sample could be recovered and tested in the onsite laboratory. Representative samples were recovered using conventional shovel sampling methods (Figure 3), an automated robotic sampling device (presented as Figure 4), and an automated cold belt sweep sampler (Figure 5). A mechanical hot-mix sample splitting device was used in the onsite laboratory to avoid rapid cooling associated with conventional quartering and its subsequent effect on laboratory sample compaction temperatures.

Construction of the actual test sections was allowed to begin after sponsors were satisfied with their trial mix results. Enough mix was produced in a continuous run to accommodate placement of both the inside and outside lanes of a single lift to minimize the amount of wasted material required to obtain stable production. Since most of the equipment was relatively cool due to the nature of the sporadic production runs, the plant was typically allowed to produce mix at a slightly elevated temperature.

Two 24-ton haul trucks were loaded and driven the short distance to the location of test section placement, with the balance of the plant run being kept in the integrated 65-ton surge bin. Paving was allowed to begin only when both trucks were lined up and ready to discharge into the material transfer device (MTD). Generally, the inside lane was paved first to establish a rolling pattern and was then utilized for destructive coring so that corrected nuclear gauge testing could be done non-destructively in the research (outside) lane.

Figure 3. Removing Shovel Sample from Truck on Roadway

Figure 4. Sampling Mix at Plant with Automated Robotic Sampler

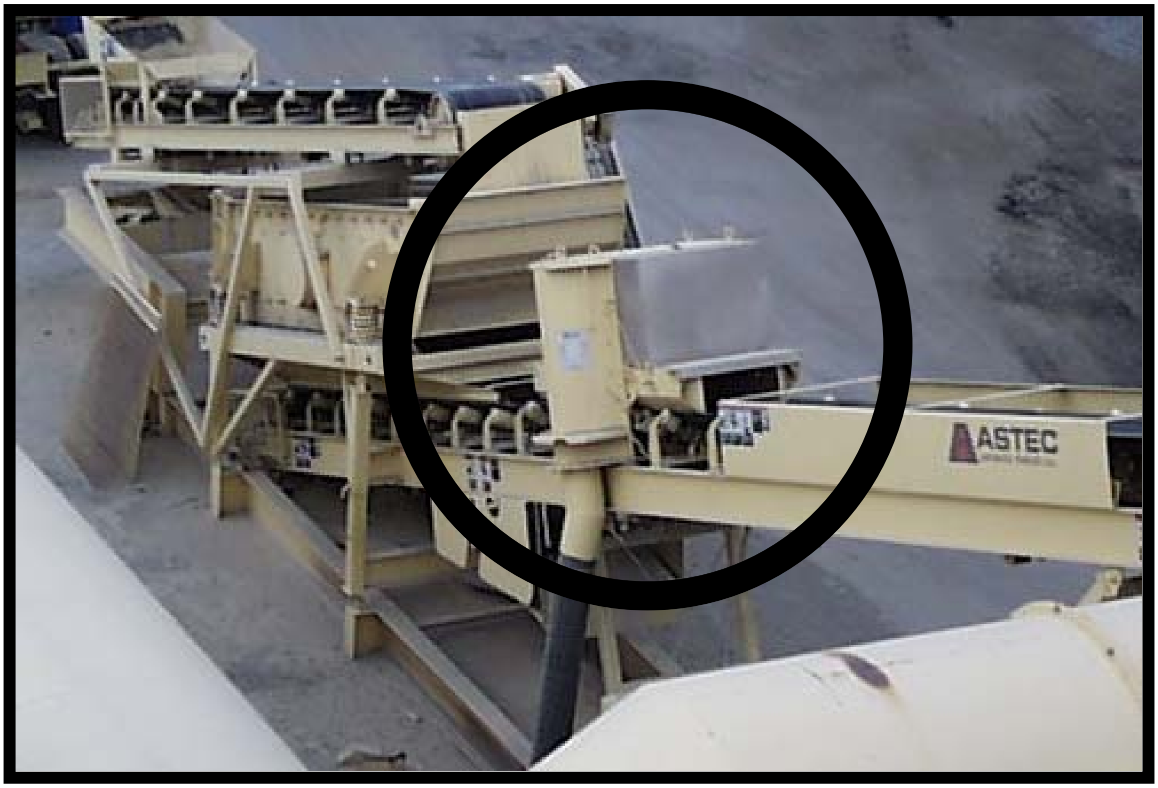

Figure 5. Automated Cold Feed Belt Sampling Device at Plant

In every case, it was required that placement operations proceed in the direction of traffic (counter-clockwise). At the far end of the section, the paver overran the joint location by 5 to 10 feet before lifting the screed. This allowed the paver to be driven clear of the immediate construction zone. Typically, two pavers (conventional and gravity feed, presented in Figures 6 and 7) were used to pave a section such that the first unit paved the inside lane and the second unit paved the outside lane.

Relative increases in density were monitored in the inside lane to identify the breakpoint in the compaction operation, which was used to establish the roller pattern in the outside (research) lane. Vibratory steel-wheeled rollers (Figure 8) were used for breakdown rolling, a pneumatic rubber-tired roller (Figure 9) was used as necessary for intermediate rolling, and the vibratory steel-wheeled roller was used in static mode for finish rolling.

Concurrently, the MTD was advanced slightly and boomed over to accommodate dumping 2 to 3 tons of blended mix into a front-end loader (Figure 10). This material was utilized for the fabrication of numerous research specimens that were later used for laboratory performance testing. When filled, the front-end loader was driven back to the onsite laboratory where material was sampled and stored in buckets for later testing (Figure 11).

Figure 6. Conventional Paver Placing Experimental Mix on Track

Figure 7. Gravity Feed Paver Used to Place Experimental Mixes

Figure 8. Vibratory Steel Wheel Roller Compacting Track Mix

Figure 9. Pneumatic Rubber Tired Roller Compacting Track Mix

Figure 10. Sampling Research Mix by Dumping into Front End Loader

Figure 11. Storing Research Mix in Metal Buckets

Once the placement and compaction operation for both lanes had been completed, a straightedge was used to identify a distance from the far end of the mat to create the transverse joint. A chalk line was then popped at this distance and a masonry saw was used to cut a clean vertical face in the new mat.

The smoothness specification was utilized to review and accept the quality of joint construction for every section on the Track. Although all joints passed their ¼ inch deviation tolerance using a 15 ft straightedge, it was later decided (based upon objective smoothness analyses) that diamond grinding should be utilized to enhance the rideability of 11 of the 46 transverse joints.

B. Instrumentation

The amount of instrumentation used on this project was minimized since the pavement structure was the same all of the way around the track. It was decided to only use moisture and temperature gauges. Any future work that may involve structural evaluations will certainly require much more sophisticated instrumentation.

As shown in Figure 12, four temperature probes were placed in every test section, within 6 m (20 ft) of the data assimilation stations. The temperature probes were installed at four different depths within the HMA, ranging from the bottom of the 150 mm (6 in.) thick upper asphalt binder course to the pavement surface. Probes placed at the bottom of the binder course and at the top of the binder course were positioned in the center of the outside traffic lane. Probes placed at the middle of the experimental mix and at the pavement surface were positioned 0.3 m (1 ft) inside the outer edge of the outside traffic lane.

Each of these computers was equipped with the Campbell Scientific software package PC208, which served as an interface to each data assimilation station. The software can call each station individually. During each data call, the computers retrieved data that was temporarily stored in the data loggers. Each data logger retrieved data for two sections as shown in Figure 13. The data acquisition computers retrieved data from the data loggers once each hour and saved the data to designated ASCII files. These probes provided data to attain minimum, maximum and average temperatures.

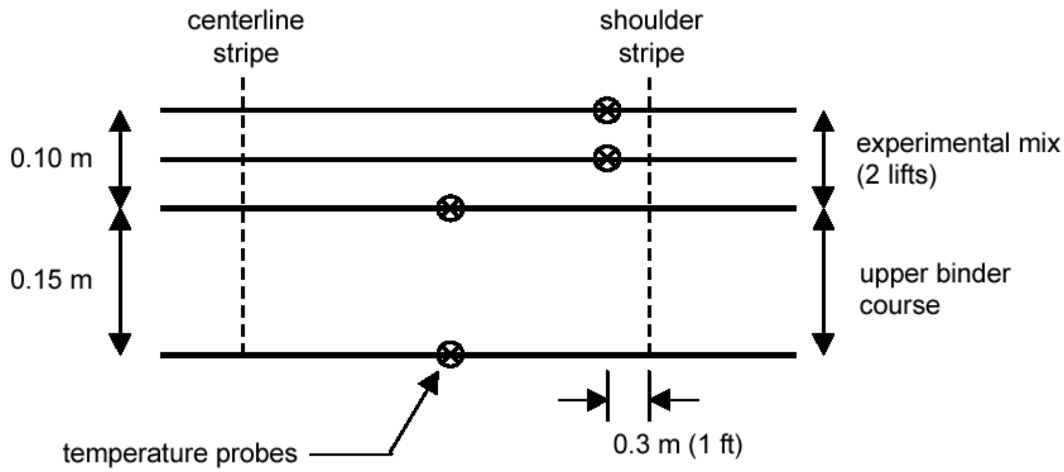

Figure 12. Layout of Multi-Depth Temperature Probes

Figure 13. Typical Datalogger Serving Two Adjacent Sections

Since experimental sections were installed at the Track in both cut and fill locations, it was considered important to document potentially varying subgrade moisture contents and consider their effect on surface mix performance over time. Dielectric gauges were selected for use based upon price, durability, calibrated accuracy, and reliability. These devices send electrical waves down slender metal antennae and are equipped to measure the length of time necessary for the waves to propagate to the “open” end of the rods and back to the controller. Analogous to high strain testing in driven piles, the waves are slowed primarily by the moisture content in surrounding soils, which results in longer travel times. Consequently, time domain reflectometry (TDR) moisture gauges were installed approximately 3 inches into uniform subgrade material directly underneath every other transverse joint. This insured that a continuous record of subgrade moisture content could be recorded at one end of every section. As with multi-depth temperature data, high-low-average summaries of subgrade moisture contents around the Track were transmitted to laboratory computer systems on an hourly basis.

VI. TRAFFIC

Four trucks haul triple trailer (tractor with 3 loaded trailers as shown in Figure 14) assemblies around the Track at 45 mph for 17 hours a day (six days a week) in order to apply 10,000,000 ESALs of traffic to the Track within two years. Although the main focus of the research is the accelerated performance of the test sections, data collected and observations made in support of the trucking operations have provided valuable information upon which future trucking operations can be refined.

Figure 14. Traffic Application via Triple Trailer Trains

The premise behind the NCAT Test Track is to apply 10 million equivalent single axle loads (ESAL's) to the test sections in just two years. It was decided that the trucking operation would be contracted out on a competitive bid. On June 15, 2000 Covenant Transport from Chattanooga, TN was selected as the contractor. The first truck rolled onto the Track on September 19th. Subsequent trucks were phased in so logistical issues could be worked out beforehand. It was also considered to be important that trucking not be applied at an accelerated rate until the HMA had a few days to age and stiffen. Full trucking was authorized to begin on November 18, 2000. The contractor supplied mechanics, drivers and all necessary equipment for the life of the two-year project.

The Track utilized four 2000 model FLD-120 Freightliner tractors with 60 Series 430 hp Detroit diesel engines to pull a series of three tandem trailers each. The Federal Highway Administration allowed NCAT the use of the trailers that were built to apply accelerated loading to experimental pavements on their project (WesTrack) at the Nevada Automotive Test Center. The gross vehicle weight of each rig was approximately 152,000 pounds. Because of the increased weight and stress on the tractors, the frames of each had to be double reinforced and equipped with a high torque drive train that could handle the higher load. In addition, Track rigs were equipped with a radar-based collision avoidance system and an infrared vehicle identification system.

The collision avoidance system consisted of sensors installed on the front and sides of the tractor. Its purpose was to monitor the proximity of road hazards to the vehicle. If the rig came within a preset limit to another vehicle the cruise control was disengaged and an alarm sounded to alert the driver of a possible collision.

The vehicle identification system was used to log laps made by each truck as it went around the Track. Attached to the front of every rig was an infrared emitter. Each emitter was set to a unique frequency such that when the truck crossed the path of the sensor, the frequency was captured by the sensor and logged onto a computer located in the Track laboratory. In this manner, ESAL counts and load spectra were calculated for every truck, thereby giving an accurate count of traffic accumulated.

As a backup to the vehicle identification system, the Track utilized driver log sheets and pneumatic counters. Each driver was required to fill out a log sheet for every shift. Each log sheet contained the date, driver's name, the truck and train number, beginning and ending mileages, number of stops made, and total gallons of fuel received to top off the truck at the end of the shift. A database was designed to house all of this information and calculate ESAL numbers and fuel consumption. The design of the database was such that trucks and trailer assemblies were monitored separately, so that if tractors had to be switched or a section of the trailer assemblies had to be taken out, the ESAL count was accurate. The pneumatic counters were located on the North tangent and the East curve. All axles were weighed before traffic began so that ESALs could be accurately calculated for all tractor-trailer combinations.

Safety was of the utmost importance at the Track. A set of safety guidelines was instituted before the first truck was allowed to operate. The drivers worked an eight to ten hour shift each day with an hour for lunch and a fifteen-minute break, but they were responsible for assessing their ability to drive. If they became too tired or sick to drive carefully, it is considered a major violation of the safety plan not to pull off the Track and rest.

There was also a protocol for entering and exiting the Track. If one truck exited the Track then all trucks must exit before the truck could enter again. The drivers made sure the ramps were clear by keeping in constant contact via hand held radios. The mechanic on duty also had a radio at all times to ensure that the drivers could relay any vital information about truck or track problems that needed to be corrected.

Due to the weight of the trailers, it took a truck almost a quarter of a mile to come to a complete stop. Because of the strain on the axles and the tires, several sets of tires were lost due to the wheel lug studs sheering off. Three axles failed due to the weight of the trailers and literally split in half (Figure 15). In consideration of these issues, no one was allowed on the Track while the trucks were running.

Figure 15. Broken Rear Axle in Last Trailer of Train

At the start of the project, it was decided that NCAT would furnish fuel for all the trucks. Through Auburn University, fuel could be purchased at lower prices because of government tax rates. This also gave NCAT the ability to closely monitor fuel consumption and how it changed over time. The cost per gallon of fuel averaged approximately $0.94 during the project. The minimum and maximum cost per gallon over the life of the project was $0.72 and $1.10, respectively. As of July 2002, the Track had used 222,761 gallons of fuel at a cost of $210,486.75.

Tire wear was been a major factor in the trucking operation. The first set of steer tires lasted only 5,000 miles, which did not compare well with conventional long haul expectations. An expert in truck alignment was brought in to identify methods to extend tire life. The consultant's recommendations resulted in numerous improvements in standard practices at the Track. Because of this concerted effort, the life of the steer tires was increased from 5,000 miles to 45,000 miles even though each truck carried approximately twice the legal gross vehicle weight (since there are no bridges on the Track, gross vehicle weight is not an issue). The life of the trailer and drive tires were also extended to 90,000 and 70,000+ miles, respectively.

Several companies have participated in research associated with the trucking operation. In addition to running tire wear experiments on both virgin and recapped truck tires (in exchange for free use of the tires), experimental fenders have been added to research their potential to reduce road spray and extend tire life. Figures 16 through 18 show road spray on three different types of pavement surface: Open Graded Friction Course (Figure 16), Stone Matrix Asphalt (Figure 17), and Superpave (Figure 18). Notice that the spray is much less on the OGFC than on the dense graded Superpave mix and the SMA mix. The spray on the SMA mix was lower than that for the open graded friction course.

Figure 16. Road Spray on OGFC Mix

Figure 17. Road Spray on SMA Mix

Figure 18. Road Spray on Coarse Graded Superpave Mix

Fuel consumption was also monitored closely. Of the four trucks used, two were equipped with full manual transmissions and two had auto shift transmissions. An apparent small difference in fuel mileage was observed between the two types of transmissions, with auto shift transmissions averaging 5.04 miles per gallon and the manual transmissions averaging 4.89 miles per gallon. Over the life of the project, average fuel consumption was approximately 5 miles per gallon.

Relative increases in density were monitored in the inside lane to identify the breakpoint in the compaction operation, which was used to establish the roller pattern in the outside (research) lane. Vibratory steel-wheeled rollers (Figure 8) were used for breakdown rolling, a pneumatic rubber-tired roller (Figure 9) was used as necessary for intermediate rolling, and the vibratory steel-wheeled roller was used in static mode for finish rolling.

VII. DATA COLLECTION

Trucking operations were suspended each Monday to allow NCAT personnel safe access to the surface of the Track to conduct pavement management studies. Field performance was documented weekly in the form of transverse and longitudinal profiles, surface texture measurements, and nondestructive density testing. Deflection testing and skid testing were conducted monthly, and cores are cut every quarter to generate correlations for nondestructive testing and to facilitate layered densification analyses.

Before construction of experimental sections had been completed, random numbers were used to identify longitudinal positions on which transverse profiles could be measured over time. Allowing 25 feet for transition into and out of each section, the middle 150 feet of each experimental mat was divided into three 50-foot statistical observations. Using 3 random numbers, a location within each observation area was identified on which transverse profiles were measured for the duration of the research. Random numbers were used as the basis of the weekly testing program as a precaution against getting in or out of phase with vehicle dynamics that may have resulted from transverse joints or bumps in the roadway.

Transverse profiles were measured weekly using a precision differential level (dipstick). Tacks were installed on the terminus of each stratified random transverse profile mark painted onto the surface of the Track (3 per section), upon which the dipstick was "walked" from the centerline in the direction of the outside edge of pavement. With each "step," the differential elevation between the feet on the ends of the device was recorded. Initially, data was recorded on paper forms and entered into computers at a later time in the office. Handheld computers were later implemented to eliminate the potential transcription and keypunch error inherent in manual data collection. In late spring of 2002, an automated version of this device that utilized a user propelled walking mechanism and automatic data acquisition was implemented that further increased the quality of transverse profile data by eliminating opportunity for human error.

Concurrently, nondestructive density testing was conducted in the wheelpaths on these same transverse profile points. Both nuclear and non-nuclear methods were employed at the Track to document wheelpath consolidation within the middle 150 (research) feet of each section. It was not desirable to cut cores from within the research portion of experimental mats because of the negative impact the extraction process would have on performance. Initially, a full set of nondestructive wheelpath density data was collected every two weeks; however, as the surface of the Track aged, it became more time consuming to obtain reliable data (e.g., more surface voids error, more difficulty in seating the device, etc.). The speed and ease of use of the non-nuclear gauge also made it possible to obtain density profiles for each section, where wheelpath densities were periodically measured every 5 feet along both the inside and outside wheelpaths.

While profiling was conducted in the transverse direction in a stratified random manner, an inertial laser profiler was also used weekly to document the longitudinal profiles of both wheelpaths. The device used for this purpose at the Track was also equipped with a high frequency laser in the passenger wheelpath to allow for characterization of the surface texture of each experimental mix over time. Additionally, a mid-lane reference laser allowed the test vehicle to record an estimate of the average rut depth via a 3-point approximation method. Where the profiling in the transverse direction provided stratified random performance data, longitudinal profiling provided continuous performance data for the entire 150-foot research portion of each section.

In support of their investment in Track research, the Alabama Department of Transportation visited the site monthly to collect surface response and performance data. Falling weight deflectometer (FWD) data was collected monthly at two locations within each section (between the wheelpaths at a point 30 and 130 feet from each transverse joint) that could be related to past testing conducted with the completion of each stage of construction of the pavement buildup. Additionally, wet skid testing was conducted with a ribbed tire to document the changes in the friction coefficient that occurred with time and traffic. Finally, coring was conducted quarterly from the inside wheelpath of the last 25 feet of each test section to provide data used to correlate nondestructive testing and document multi-layer consolidation over time.

All verified data was ported into an Access database from which Adobe reports were generated and posted on the project web site (www.pavetrack.com). Since the locations of the wheelpaths must be known to accurately compute rutting via transverse profiles, continuous 3-point approximations have served as the basis of the project's historical web record for each section's rutting performance over time (see the "performance" page and click on any section to view performance data). With transverse profile data using both manual and automated methods from two summers now complete, rutting via stratified random transverse profiles was computed and served as the basis of field comparisons and lab-to-field correlations for the final project record.

VIII. FINDINGS

A. General

The findings provided at this point are preliminary since the traffic had not been completed at the time this report was written. More detailed final reports will be provided at a later date. The primary purpose of this report was to highlight the observations, made to this point in time, related to design, construction, and performance of the track.

There were a total of 46 test sections constructed using various aggregates, grades of asphalt, and various mixture types. Some mixtures were designed with marginal aggregates and some mixtures were designed with 0.5% additional asphalt. Several mixture types were used including fine and coarse graded Superpave, stone matrix asphalt, open-graded friction courses, as well as some variations of these mixtures. After over 9 million ESALs had been applied, the most amazing thing about this entire study was that very little rutting had occurred in any of the sections. The track was designed to be sufficiently strong so that fatigue cracking would not occur resulting in rutting as the expected form of distress. The average rutting at the track was approximately 0.12 inches after approximately 9 million ESALs. Rutting is typically not considered to be a problem until the magnitude reaches approximately 0.5 inches so the rutting observed at the track was minimal. The two test sections with the most rutting (approximately 0.25 inches) were sections that did not use a modified asphalt and in which an additional 0.5% asphalt binder was added.

Several topics are discussed below related to the observations made during the track operation. Some of the observations are related to pavement performance, trucking, and construction but all are believed to be important to the success of the track.

B. Analysis of Temperature Data

Introduction

As the NCAT Test Track was constructed, temperature probes were installed at various depths within the pavement layer. There were 184 temperature probes installed with four probes installed for each of the 46 test sections.

The Datalogger received temperature data every minute and then recorded the minimum, maximum, and average pavement temperature every hour. This meant that each Datalogger received more than 11,500 temperature inputs per day (1). Of the 184 probes, problems were experienced in recording erroneous day, hour, or temperature data from eight gauges in the south loop and 24 gauges in the north loop. Therefore, about 17 percent of the gauges were deficient in recording data.

Pavement Maximum Temperature

One of the criteria for designing Superpave hot mix asphalt (HMA) is to select asphalt binder grades based on the seven-day highest average temperature of the pavement at a depth of 20 mm. This temperature is then related to ambient temperature for convenience. In 2001, the highest seven-day air temperatures were from July 6-July 12 (day 187-193) and averaged 33.52°C (92.34°F). In 2002 the highest seven-day air temperatures were from July 15-July 21 (day 196-202) and averaged 34.63°C (94.33°F) based on data from the weather station located at the Track.

Since the maximum seven-day average air temperature was known from the Track weather station and the latitude of the Track is 32.6 degrees, the pavement surface temperature and pavement temperature at 20mm depth could be calculated using Superpave temperature equations (2). Based on this information, the maximum average pavement surface temperature was calculated to be 59.9°C (139.8°F) during the seven-day period in 2002 when air temperatures were the highest. By using the Track latitude of 32.6 degrees, the maximum pavement surface temperature could also be calculated by simply adding 25.3°C (45.5°F) to the maximum air temperature. From the surface gauges at the Test Track, the average maximum surface temperature for the test sections was 61.4°C (142.6°F). The predicted surface temperature was 59.9°C (139.8°F). The calculated and measured surface temperatures compare very favorably.

Temperature Vs. Pavement Depth

It is well known that pavement temperatures rise and fall due to diurnal and seasonal variation. The average high temperature of the pavement surface increased about 33°C (60°F) from the coldest month of the year to the hottest month. The temperature at the bottom of the binder course increased about 28°C (50°F) during this same period. The temperature at the bottom of the binder layer remained above 27.7°C (80°F) for 24 hours per day from May through September. During the hottest month of the year (July), the temperature at this depth of 250 mm (10 in) reached 39.1°C (102.4°F). The temperature at the bottom of the binder layer approximately 250 mm (10 in) below the surface remained relatively constant (about 5.5°C (10°F) variation) over a 24-hour period. By comparison, the surface temperature may vary as much as 28°C (50°F) during the same 24-hour period. There was a time lag involved for the high temperature to penetrate and transfer to the lower layers. The pavement surface temperature was highest around 2:30 p.m., but the temperature at the bottom of the binder layer did not reach its maximum temperature until around 10:00 p.m., nearly eight hours later.

Effect of Mix Type on Pavement Temperature

Several mixture types were in place at the NCAT Test Track. It was desirable to examine the effects these various mixture types may have on temperatures within the pavement.

To examine the effect of mixture types, a comparison was made of temperatures at various depths for the OGFC, SMA, and Superpave sections. The layer interface beneath the OGFC (at the middle of the research layers) in July 2001 was 1.7°C (3.1°F) cooler than for the SMA and 2.1°C (3.8°F) cooler than when Superpave surface mix was used. In 2002, the temperature beneath OGFC and SMA surface mixes was virtually the same at 53.7°C (128.6°F) and 53.6°C (128.5°F) respectively, while the temperature under the Superpave surface was 55.1°C (131.2°F), a difference of 1.4°C (2.6°F). Therefore, it appears the open surface texture of OGFC and SMA mixes may allow underlying mixes to be slightly cooler than when conventional dense-graded surface mixes are used.

C. In-Place Densification Results

The NCAT Test Track offers a unique opportunity to study pavement densification and its relationship to the number of design gyrations, since all of the sections received the same traffic, had the same base and subgrade support and were exposed to the same climatic conditions. Thirty-two of the test track sections were designed using Superpave and were included in the following analysis. The 32 sections represented a range of aggregate types, nominal maximum aggregate sizes (NMS), and gradations. Primarily one compaction effort, N(design) = 100 gyrations, was used to design the sections. However, two sections were designed with N(design) = 125.

One of the objectives of the work at the track was to evaluate densification of HMA. Cores, for evaluating densification, were taken at various traffic levels from the left wheel path of each section. Initially, traffic began in September 2000 with only one truck in operation and traffic was fully implemented in February of 2001. For the first three months, cores were taken on a monthly basis and later quarterly. The cores were sawed into their respective layers and the bulk specific gravity of each layer determined using AASHTO T-166. Density of samples having greater than 2 percent water absorption was determined using the Corelok Device (3). Densities of the cores were calculated using the construction maximum specific gravity values. Figure 19 shows the average test track pavement density as a function of ESALs for the Superpave sections through September 2002. The figure indicates that the initial construction densities were slightly lower for the PG 76-22 surface layers as opposed to the other layers. For both the PG 67-22 and PG 76-22 sections, the construction densities were less for the upper lift. The data seems to indicate distinct rates of densification for each lift/binder combination related to time after construction and temperature (season). There appears to be an initial seating of the mix between the first and third data points taken in September and December of 2000, respectively. The average pavement density appears to continue to increase from December 2000 (third data point) through October 2001 (data point at approximately 4.5 million ESALs). There is little increase in pavement density between October 2001 and June 2002 (data point at approximately 7.5 million ESALs). In fact, the average density for all but the PG 67-22 upper lift sections appears to decrease in March 2002 (data point at approximately 6.5 million ESALs). The change in density during the summer of 2002 (7.5 to 8.5 million ESALs is similar to that which occurred during the summer of 2001 (3.0 to 4.5 million ESALs).

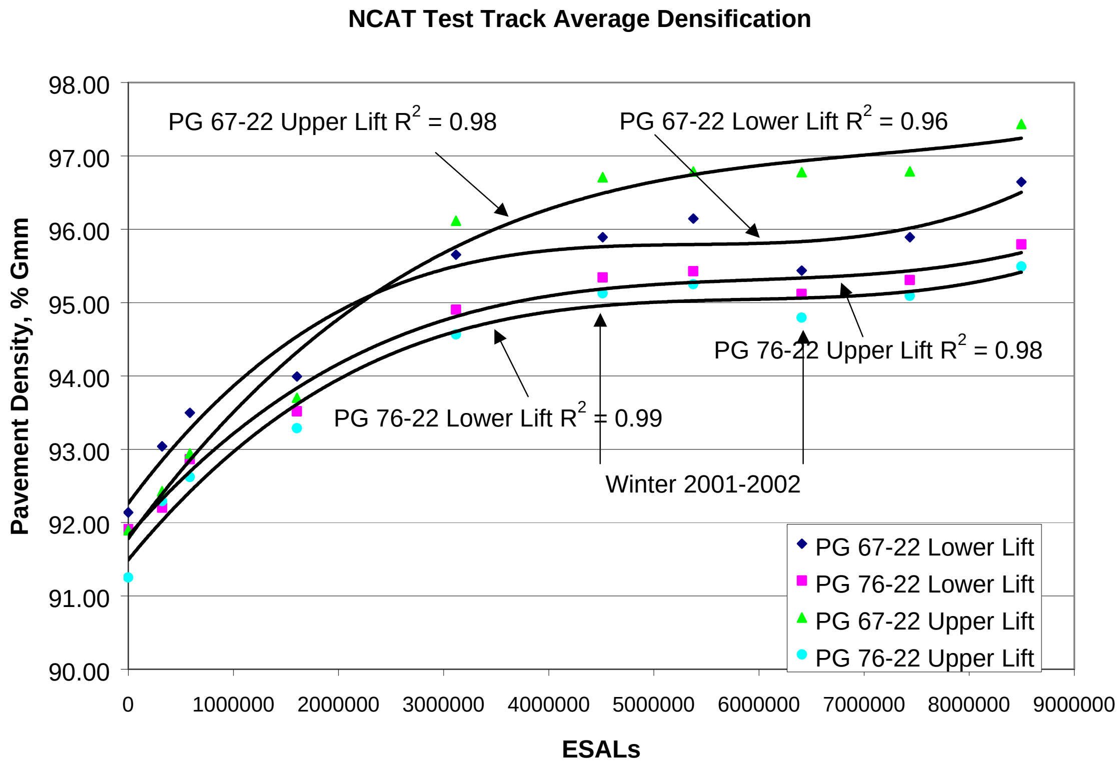

Figure 19. Average Test Track Pavement Densification

There appears to be a significant difference in the rate of densification based on binder grade. As expected, the sections with a softer binder, PG 67-22, densified faster. This is true for both the upper and lower lifts. Further, it appears that for the PG 67-22 sections, the lower lift, which is 50 mm (2 inches) below the surface of the pavement, did not densify as fast as the PG 67-22 surface lift. The difference in density is approximately one percent from approximately 3.0 through 8.5 million ESALs. The difference is not apparent prior to 3 million ESALs because the lower lifts were constructed at a higher initial density.

All QC samples were compacted using the same Troxler Model 4141 Superpave Gyratory Compactor (SGC). The Troxler Model 4141 SGC was the same brand and model as that used for the majority of the mix designs. Three replicate samples were compacted for each sublot. The samples were compacted to the same N(design) level used in the mix design, generally 100 gyrations. The bulk specific gravities of the samples were determined with AASHTO T166. Back calculating the bulk specific gravity data for various gyrations provided density results at all applicable compaction levels.

Equation 1 can estimate the density at any gyration level.

$$Density\ at\ Gyration\ X\ = Density\ at\ NDesign \times \frac{Height\ at\ NDesign}{Height\ at\ Gyration\ X} \qquad (1)$$

The average predicted gyration levels are shown in Figure 20 as a function of layer and binder grade. The high R^2 values result from averaging the predicted gyrations for each binder grade and lift. Regression analysis on the entire data set produces fair to poor correlation's due to scatter in the data for the individual sections.

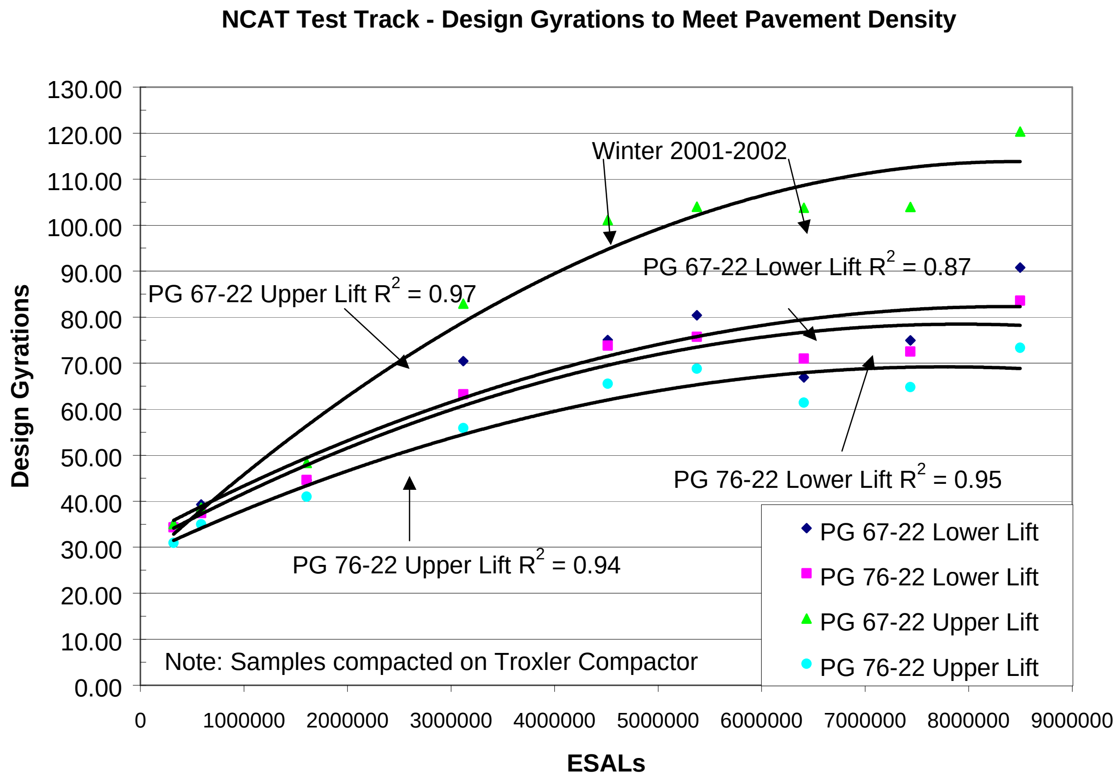

Figure 20. Average Predicted Gyrations to Meet Field Density Versus ESALs

From Figure 20, it appears that there is a significant difference between the predicted gyrations for the upper lift (top 50 mm [2 inches]) of the mixes containing PG 67-22 and both the lower lift of the mixes containing PG 67-22 and both lifts of the mixes containing PG 76-22. According to current specifications, N(design) = 75 is specified for design traffic levels of 0.3 to 3 million ESALs and N(design)=100 is specified for 3 to 30 million ESALs. From the figure, it appears that an N(design) value of 75 gyrations may be appropriate for up to 3 million ESALs for the surface layer with PG 67-22 binder. PG 67-22 meets the climatic requirements for the test track. It appears that for greater than 5 million ESALs, a compactive effort greater than 100 gyrations may be warranted with the Troxler Model 4141 SGC used at the NCAT test track. However, for other brands of compaction, the 100 gyrations may be enough. One also has to realize that more densification is likely to occur under accelerated loading than normal loading because of the difference in aging of the HMA.

Figure 20 indicates that the predicted design gyration level for PG 76-22 to meet a given field density, normally 96 percent of theoretical maximum density since one normally designs mixes for 4 percent air voids, could be lower than for PG 67-22. PG 76-22 represents a one and one half grade bump. Since densification is decreased with the stiffer binder, it may also be desirable to consider increasing the asphalt content slightly to promote more durability since densification, and likely rutting will be reduced when the higher PG grade is used.

D. Observed Rutting Compared to Densification

Significant rutting has not been observed at the NCAT Test Track. It has been suggested that the observed vertical deformation is not shear flow rutting, but instead the result of consolidation. To examine this possibility, the amount of vertical deformation (consolidation) that would be expected based on the change in density between the time of construction and the application of 8.5 million ESALs was determined. The deformation was calculated separately for each lift (upper 50 mm and lower 50 mm) and summed. The expected deformation is plotted versus the observed ARAN rutting after 8.5 million ESALs (Figure 21).

From the figure, it can be seen that the majority of the points are below the line of equality indicating the observed ARAN rutting is generally less than what would be expected solely from the amount of pavement densification that has occurred. Only one point indicates more observed ARAN rutting than predicted consolidation. This data seems to indicate that densification is the primary cause of rutting at the Track.

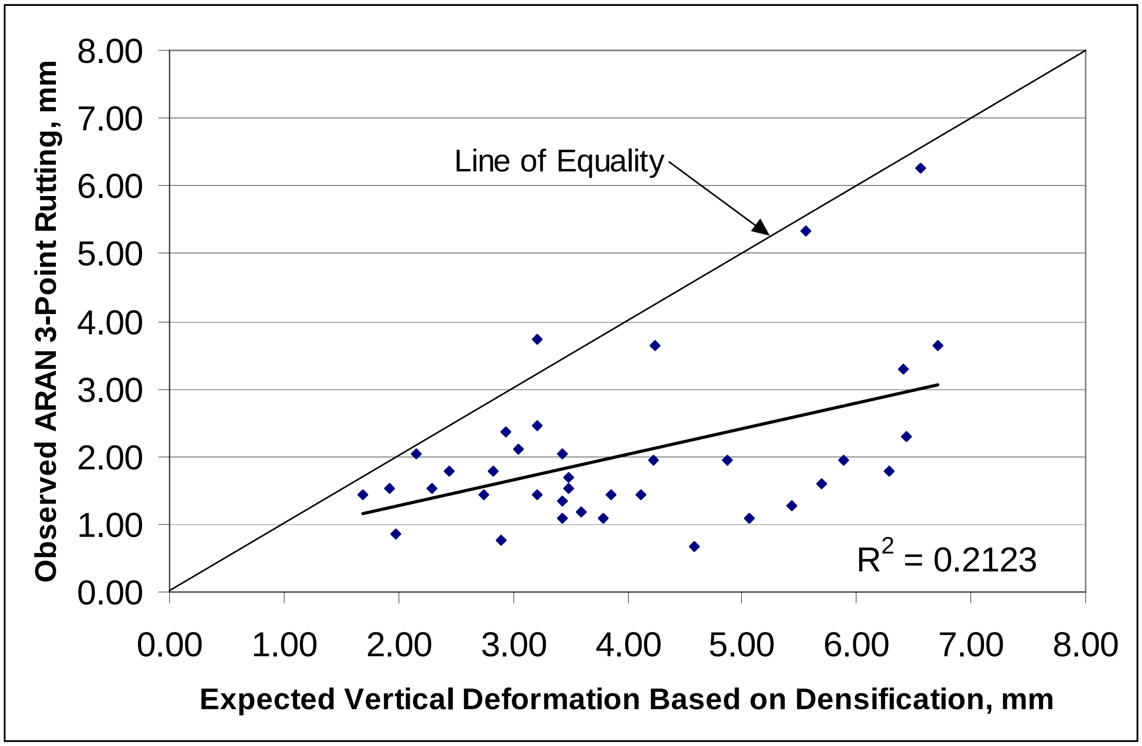

Figure 21. Expected Vertical Deformation Versus Field ARAN Rut Depths

E. Diamond Grinding to Improve Smoothness of Transverse Joints

Pavement smoothness is the number one expectation of the traveling public. Agencies are increasing the application of smoothness specifications to highway construction. Many of these specifications include strong disincentives and/or incentives. Initially, agencies used 0.10 miles (160 m) or longer sample sections to calculate pavement roughness using the International Roughness Index (IRI). Use of longer sample sections tends to average out the roughness of the pavement such that the roughness associated with a transverse cold joint or dip in the screed due to an insufficient head of material may not be apparent. With advances in technology, many agencies are now using shorter sample sections such as 0.01 mile (16 m). These shorter sample sections are capable of detecting pavement "bumps."

Diamond grinding is similar to cold milling except diamond grinding equipment uses diamond saw blades that are gang mounted to a cutting head instead of carbide teeth. This allows more precise profiling. The cutting heads are typically 36 to 37 inches wide (914 to 940 mm), though the cutting heads on new equipment may be up to 47 inches (1194 mm) wide. The diamond saw blades are spaced to provide a corduroy texture in the ground pavement. The groove spacing varies from 50 to 60 grooves per foot (164 to 194 grooves per m). Typically, the grooves are approximately 0.06 in (1.5 mm) deep (*4*). Water is used to cool the diamond blades during grinding. Diamond grinding machines utilize integrated wet vacuums to remove the grinding slurry. A typical diamond grinding operation is shown in Figure 22 (NCAT Test Track November 2000).

Figure 22. Typical Diamond Grinding Operation

In order to improve smoothness by eliminating "bumps" in the pavement, some agencies are specifying or considering diamond grinding HMA pavements. While traveling for various field projects, NCAT staff have observed diamond grinding in Colorado, Michigan, Nevada, Tennessee and Utah.

IRI measurements, made with an ARAN van and Alabama Department of Transportation's California profilograph immediately after the construction of the track, indicated that there were apparent “bumps” at several of the transverse cold joints between sections. There was concern that these bumps might cause dynamic loading on the track section following the joint.

The transverse construction joints were evaluated for all of the track sections. The evaluation included: the average IRI within ± 25 feet (7.6 m) of the joint, the maximum 5 foot (1.5 m) IRI in the vicinity of each joint and a subjective ride comfort rating from 1 to 3. The IRI measurements were determined with Alabama Department of Transportation’s California Profilograph using a 0.20-inch (5 mm) blanking band. In addition, measurements were made with a two-foot (0.6 m) straight edge to estimate the height of the high spots. Diamond grinding can be used on dips in the pavement surface, but it requires grinding over a much larger area. Figure 23 shows an example joint profile from the California Profilograph. The vertical scale is exaggerated.

Based on this evaluation, eight joints were initially identified for grinding: N8-N9, N9-N10, N11-N12, N12-N13, S1-S2, S5-S6, S6-S7 and S9-S10. Typically, grinding was performed within ± 0 to 30 feet (0 to 9.1 m) of the joint. Two sections, N9 (joint from N8) and N11 (joint leading into N12) were ground to 36 and 50 feet (11 and 15 m), respectively to accommodate dips in the as-constructed joint. Because the grinding of the initial eight joints was so successful, three additional joints were treated: N5 to N6, N7 to N8 and S3 to S4. All of the ground sections were left unsealed. A typical surface texture of a ground section is shown in Figure 24.

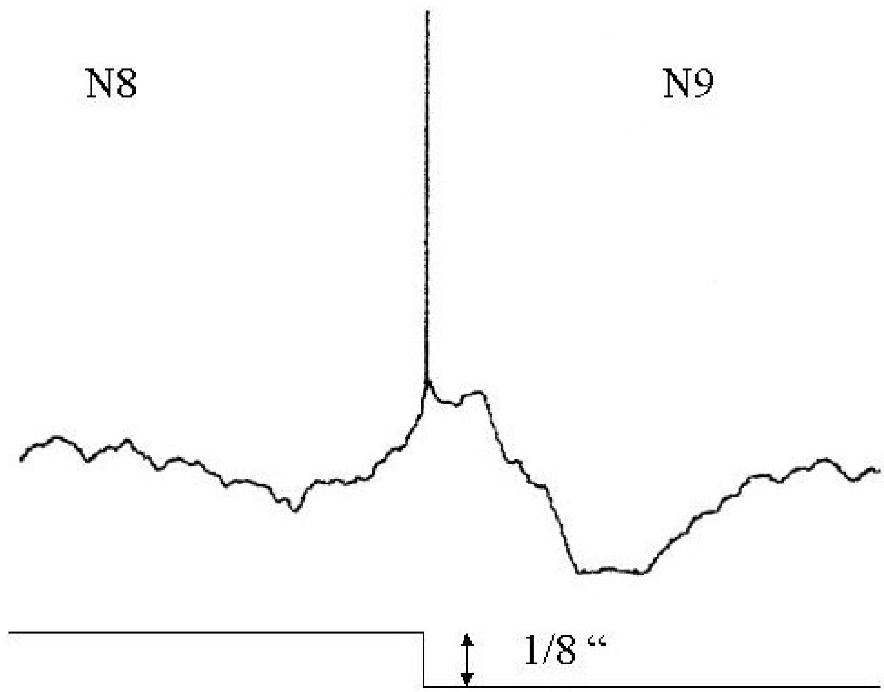

Figure 23. Example California Profilograph Joint Profile

Figure 24. Typical Ground Joint Texture

Before and after testing, measurements for IRI were also made with an ARAN van. IRI measurements were calculated using 25 ft (7.6 m) intervals. Figure 25 shows the improvement in IRI for the 25 feet (7.6 m) of pavement encompassing the joint. Diamond grinding reduced the measured IRI by 19 to 63 percent for the eleven sections treated. The average improvement was 45 percent. Overall, diamond grinding the eleven joints improved the average IRI for the test track (1.7 miles) from 68.7 inches per mile (1.08 m/km) to 65.9 inches per mile (1.04 m/km).

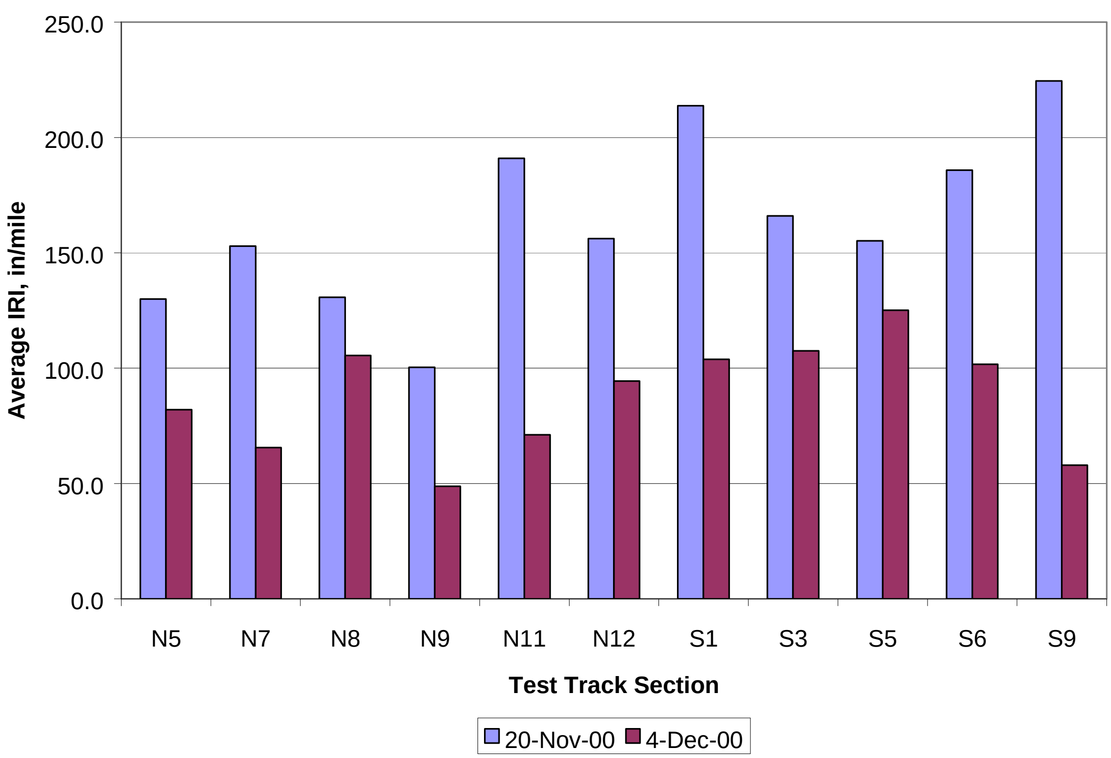

Figure 25. Average IRI Over Joint, Before and After Diamond Grinding

Two concerns with diamond grinding HMA pavements for smoothness are the long-term durability and friction properties. Visual surveys were made of the ground joints in May 2002 after the application of approximately 7 million ESALs. Two sections, S2 and N11 exhibited a minor loss of surface aggregate in the ground areas. Section S2 contained chert gravel. The corduroy texture was still plainly evident in all of the sections.

Overall, it appears that diamond grinding the transverse joints at the NCAT Test Track was a success. Diamond grinding greatly improved the IRI measurements on the joints. The grinding did not appear to affect the durability of the ground surfaces, even without the application of a seal to the ground areas. Reductions in macro texture were noted for some sections.

F. Changes in Smoothness Over Time

Efforts were made to document the changes in smoothness (or roughness) over time on the NCAT Test Track. Smoothness measurements were obtained using a Roadware ARAN van. This equipment uses lasers and accelerometers to equate the pavement's profile to an International Roughness Index (IRI).

Figure 26 illustrates the average NCAT Test Track IRI value versus the number of applied ESALs. IRI values within this figure include transverse joints throughout the Test Track. There are two initial observations about the data shown in Figure 26. First, there was a marked decrease in IRI immediately after the diamond grinding operations on some transverse joints (described previously). The average IRI for the entire track dropped by approximately 3 in/mile after the diamond grinding. A second observation is that there appears to be some increased variability in IRI measurements from approximately 1.6 to 6.3 million ESALs. These measurements represent the time period from March 2001 to March 2002. Prior to March 2001, the trend in the data was as expected in that IRI appeared to be decreasing slightly after traffic was placed on the Test Track. This would occur as the traffic smoothed out any minor construction related roughness. After about three months and approximately 500,000 ESALs, the roughness of the Test Track began to increase. The data after March 2002 also shows a trend toward increasing roughness over time. As more ESALs were applied, the roughness increased. The increased variability in IRI data between March 2001 and March 2002 was likely caused by a voltage problem within the ARAN van.

Figure 27 shows a comparison in IRI values between three Superpave gradations. This figure is a frequency diagram of IRI measurements from the beginning of trafficking through approximately 9 million ESALs. The three test sections were S9, S10, and S11. All three sections utilized a granite aggregate. IRI values shown in Figure 27 are the average IRI within the middle 150 ft of each test section. This figure shows that there were some minor differences between the average IRI values. The mixtures having a Superpave gradation passing below the restricted zone (BRZ) had the lowest average IRI value followed by the Superpave gradations passing above the restricted zone (ARZ). The gradations passing through the restricted zone (TRZ) had the highest average IRI value.

Figure 28 shows a comparison between IRI values for three different mix types. The sections utilized in Figure 28 were E4, E9, N12, and W5. Mixes included in the figure were SMA, OGFC and Superpave. Two Superpave mixes were included: BRZ and ARZ gradations. All four mixes were comprised of single source granite aggregate. Based on the figure, the SMA and OGFC mixes had collectively lower IRI values than the Superpave mixes. Based on the data in Figure 28, the SMA and OGFC mixes had approximately the same level of roughness. For the two Superpave mixes, the BRZ gradation section again had slightly lower IRI values than the ARZ section.

Based on the data presented, all of the test sections (including those not shown) exhibited relatively low IRI values. Therefore, the average IRI values shown in Figure 26, which includes all of the transverse joints, illustrate that the rideability of the Test Track was good. Most test sections had low IRI values indicating that there is no significant difference in the smoothness that can be obtained with different mix types.

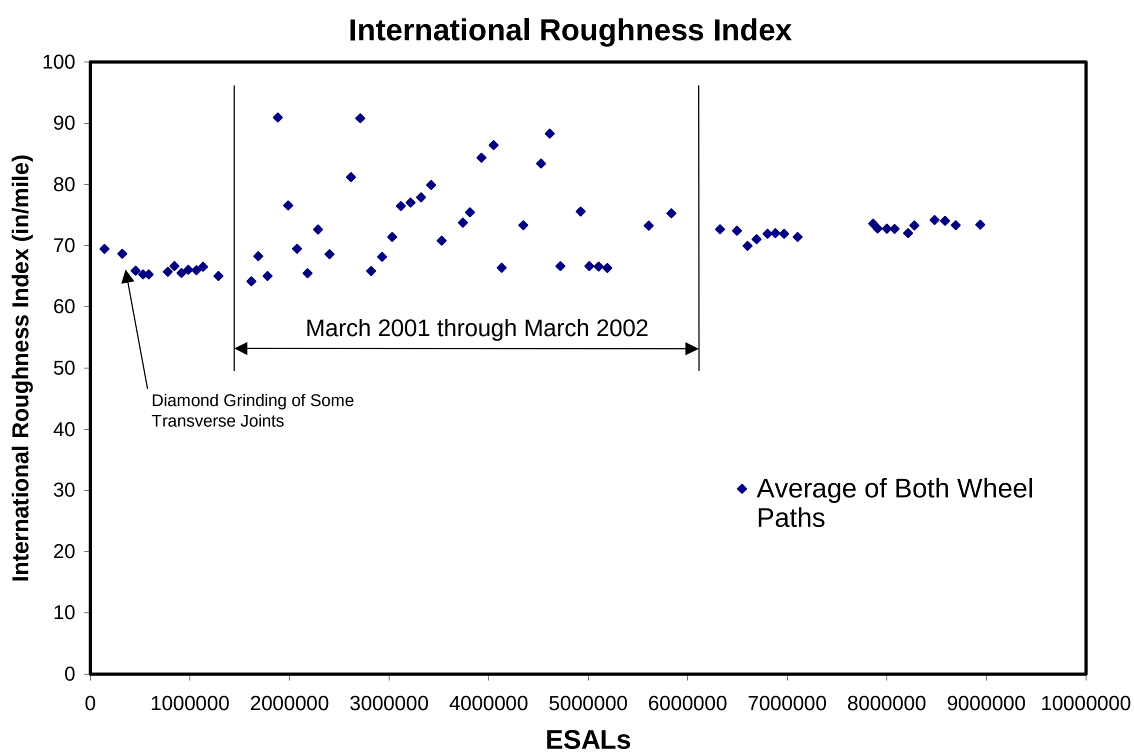

Figure 26. Average Track Roughness Over Time

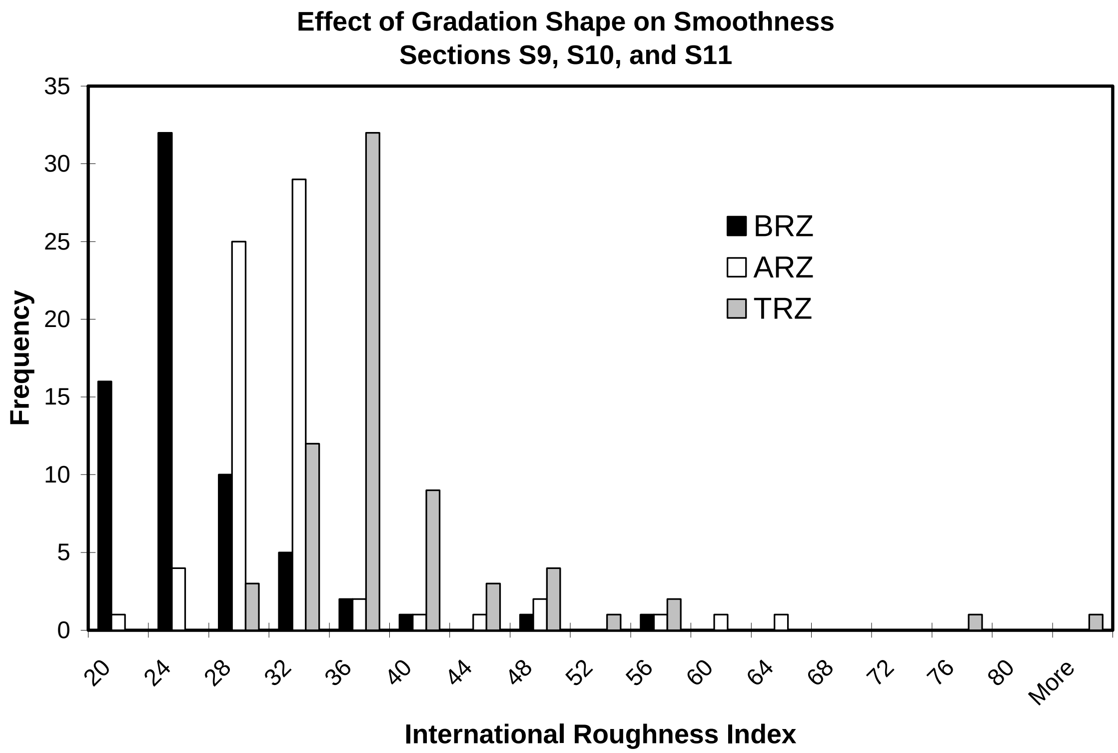

Figure 27. Comparison in IRI Values for Superpave Mixes

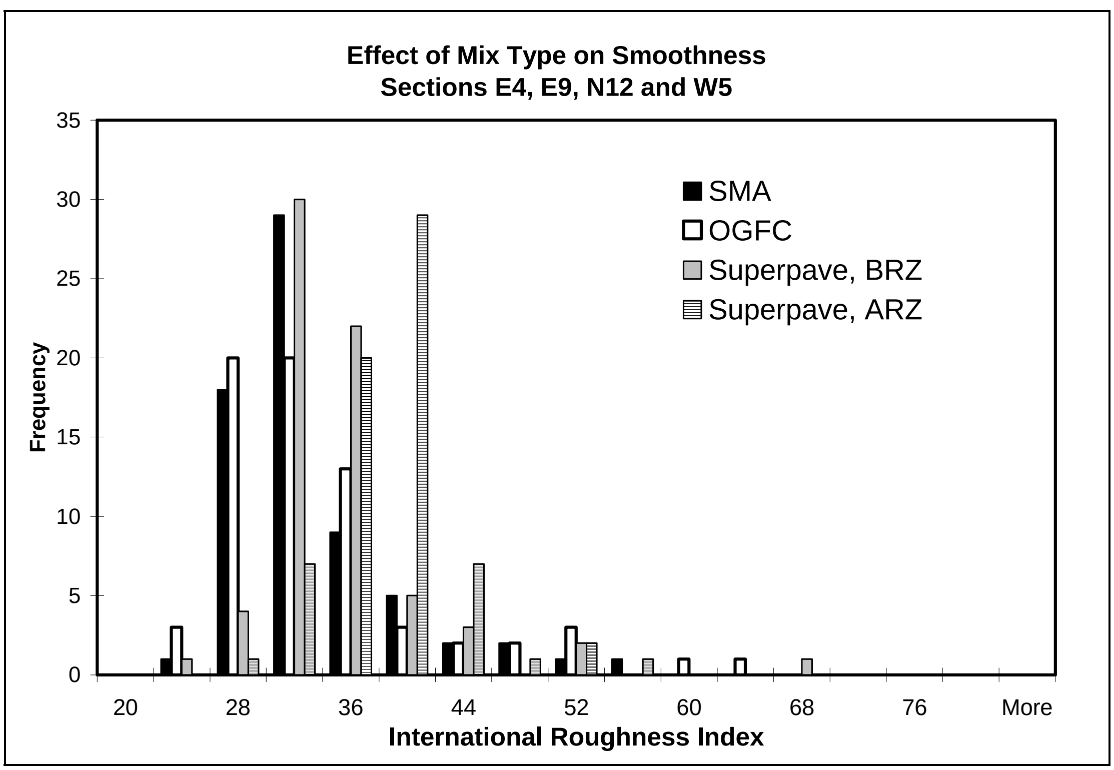

Figure 28. Comparison Between IRI Values for Different Mix Types

G. Friction Properties

Pavement friction during wet conditions continues to be a major safety concern for pavement design and maintenance. Friction is defined as the relationship between the vertical force and horizontal force developed as a tire slides along the pavement surface. Its magnitude mainly depends on the pavement surface characteristics and vehicle characteristics. Vehicle characteristics, such as speed, braking system and tire condition are not within the control of the highway road engineer. However, the highway engineer should provide a pavement surface with sufficient friction to meet the design criteria.

The friction of a pavement surface is a function of the surface texture which includes microtexture and macrotexture. Microtexture provides a gritty surface to penetrate thin water films and produce good friction between the tire and the pavement. Macrotexture provides drainage channels for water expulsion between the tire and the pavement thus allowing better tire contact with the pavement to improve friction in wet weather and to prevent hydroplaning. Currently there is no system capable of measuring microtexture profiles at highway speeds. Therefore, microtexture is evaluated by using pavement friction at low speeds as a surrogate. The classic measure of pavement macrotexture is a volumetric method, typically referred to as the "sandpatch" method (ASTM E965) (*6*). With the significant advances that have been made in laser technology and data processing, systems are now available to measure macrotexture at traffic speeds.

Previous work has indicated that skid resistance and texture are influenced by aggregate properties and gradation (*7, 8, 9*). However, it can be difficult to evaluate the effect of these properties on the measured skid resistance and texture since both skid resistance and texture will vary as traffic is applied to the pavement and due to environmental factors. The Test Track offered a unique opportunity to evaluate these factors under uniform conditions.

Figure 29 shows the relationship between SN measured at 64 km/hr with a ribbed tire and the International Friction Index (F60 after 6.44 million ESALs). As expected, there is an excellent correlation between the two measures ($R^2 = 0.97$). The ribbed tire skid number is consistently higher than F60. The slope of the regression line (0.6637) indicates the difference between IRI and SN increases with increasing SN. However, the strength of the relationship suggests that SN may be used to monitor trends in F60 with time.

Different sections at the Track represent a range of aggregate gradations. The percent passing the 2.36 mm (No. 8) sieve ranges from 13 to 54 percent. Both 9.5 mm and 12.5 mm nominal maximum aggregate size (NMAS) mixes were used. For analysis, the gradations were divided into five ranges: above the Superpave restricted zone (ARZ), through the Superpave restricted zone (TRZ), below the Superpave restricted zone (BRZ), stone matrix asphalt (SMA) and open graded friction course (OGFC). The restricted zone used for the determination was the appropriate one for the NMAS for a given section. Figure 30 shows the average ribbed tire skid numbers at 64 km/hr with traffic for the five gradation types. Agencies sometimes receive calls from concerned motorists regarding new SMA and OGFC sections. The traveling public sometimes perceives these mixes as being slippery when new, due to the high asphalt binder film thickness typical of such mixes. As shown in Figure 30, SMA and OGFC mixes do typically start off with slightly lower, but still more than adequate, skid numbers due to the high asphalt film thickness. As the binder wears off the exposed aggregates, the SMA mixtures maintain a higher skid number with traffic than the Superpave or OGFC mixtures. Though both the SMA and OGFC mixes indicate a slightly lower average skid number than the ARZ and BRZ Superpave mixes, the treaded tire skid number does not account for the increased macrotexture these mixes provide. OGFC provides demonstrated reduction in spray and hydroplaning by channeling water away from the tire/road interface. The high macrotexture of SMA provides similar benefits to a lesser degree. The TRZ sections, typical of mix gradations prior to Superpave, have the lowest skid numbers with time for all of the mixes. This may indicate the influence of mixture volumetric properties, such as VMA, on the measured skid resistance. All five of the mix types still maintained adequate friction after approximately 7 million ESALs.

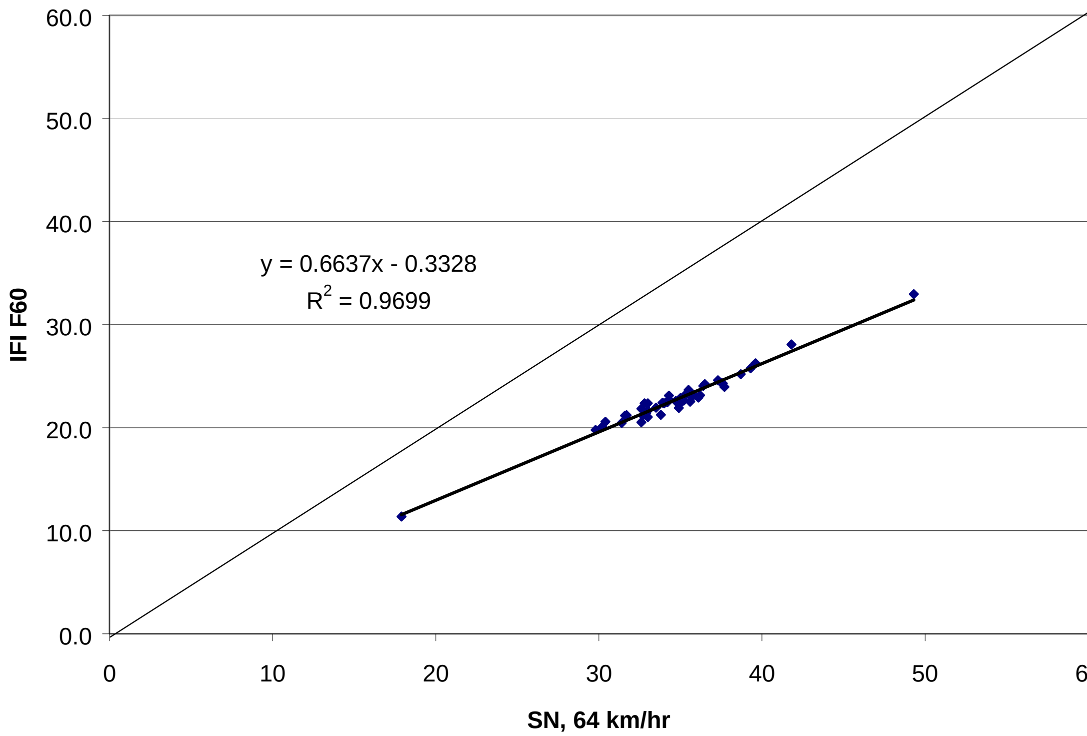

Figure 29. International Friction Index versus Skid Number at 6.44 million ESALs

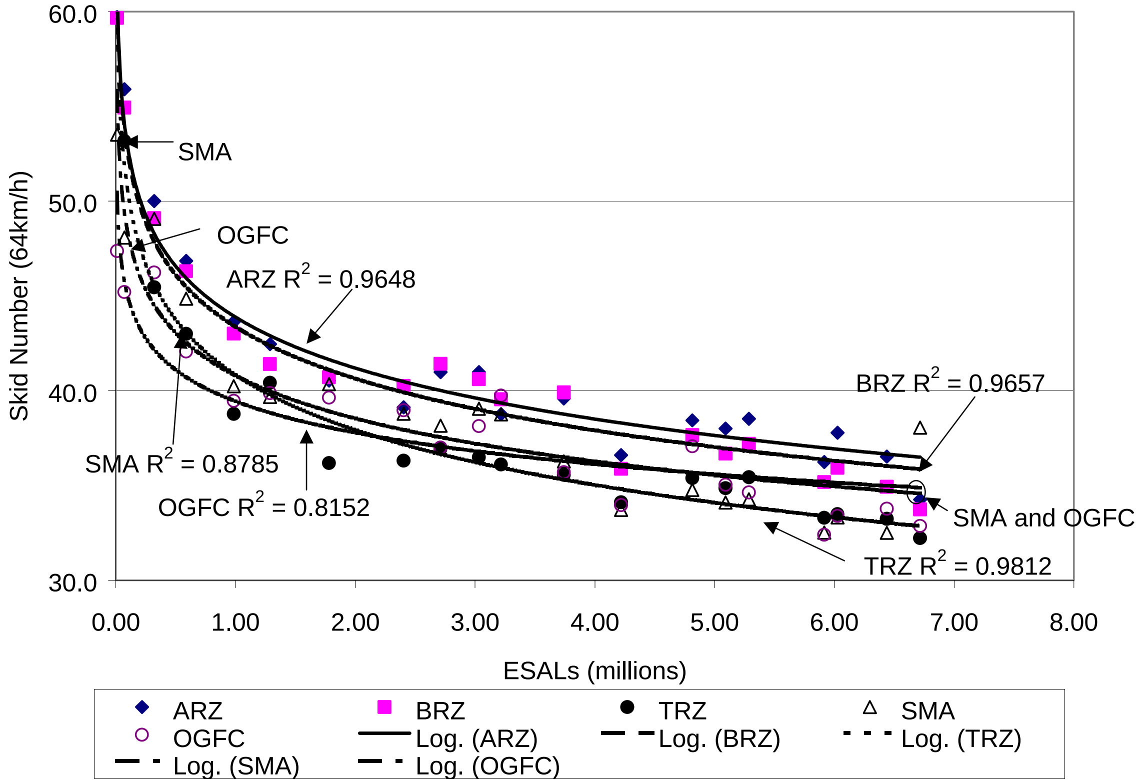

Figure 30. Ribbed Tire Skid Number versus ESALs by Gradation Type

Figure 31 shows the mean profile depth (MPD) measured with the ARAN van after 4.81 million ESALS versus the percent passing the 2.36 mm (No. 8) sieve. Analysis of variance (ANOVA) was performed using Minitab statistical software to test the significance of gradation shape on the measured MPD. The analyses indicated that gradation shape was significant when comparing the MPD of 46 test track sections at the 95 percent confidence level for all of the testing intervals. The confidence intervals for the data indicate a consistent ranking for the mixes of OGFC, SMA, BRZ, TRZ and ARZ.

Figure 32 shows a plot of the average MPD by gradation type with time. The average MPD values for the ARZ and TRZ sections indicate a slight increase with time. This may be due to a slight loss of surface aggregate. Both, the SMA and OGFC sections indicate a reduction in MPD with time. This is most likely due to a reorientation of aggregate particles under traffic. The three dates in Figure 32 represent construction, 4.81 and 6.44 million ESALs applied respectively. The BRZ section indicates on average, no change with time. An ANOVA was performed using MPD as the response variable and both Gradation shape and test date as factors. As shown previously, gradation type was significant at the 95 percent confidence level. Test date was not significant with a calculated p-value of 0.669.

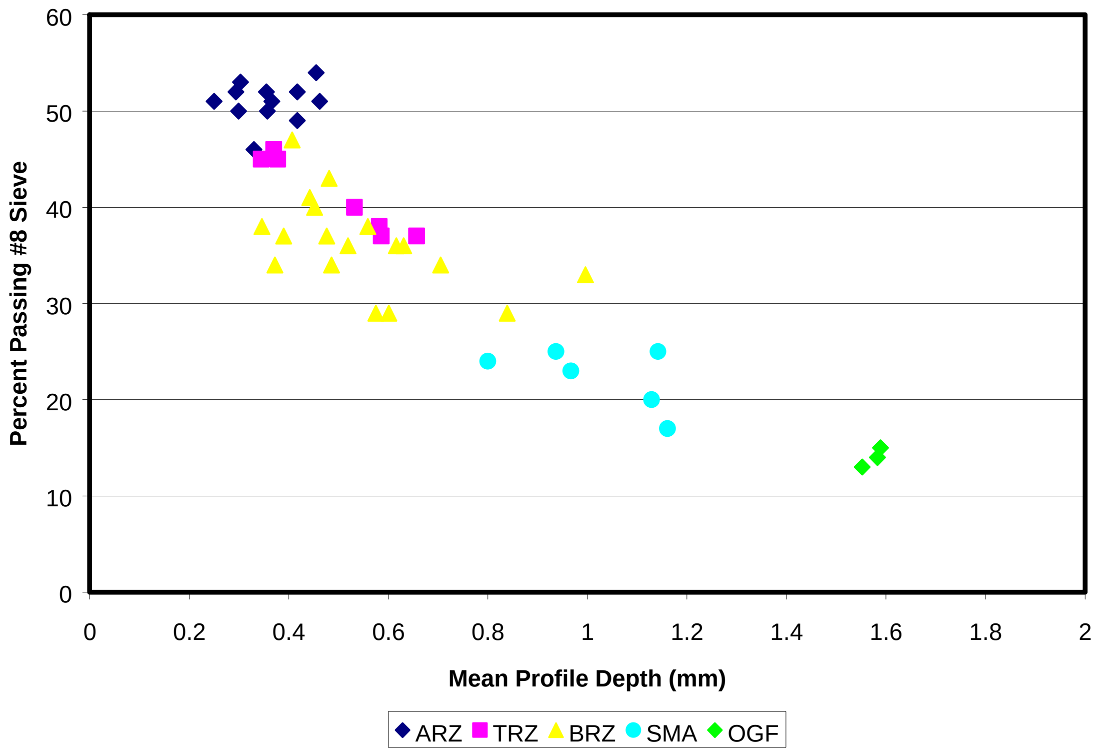

Figure 31. Mean Profile Depth Versus Percent Passing the 2.36 mm Sieve after 4.81 million ESALs

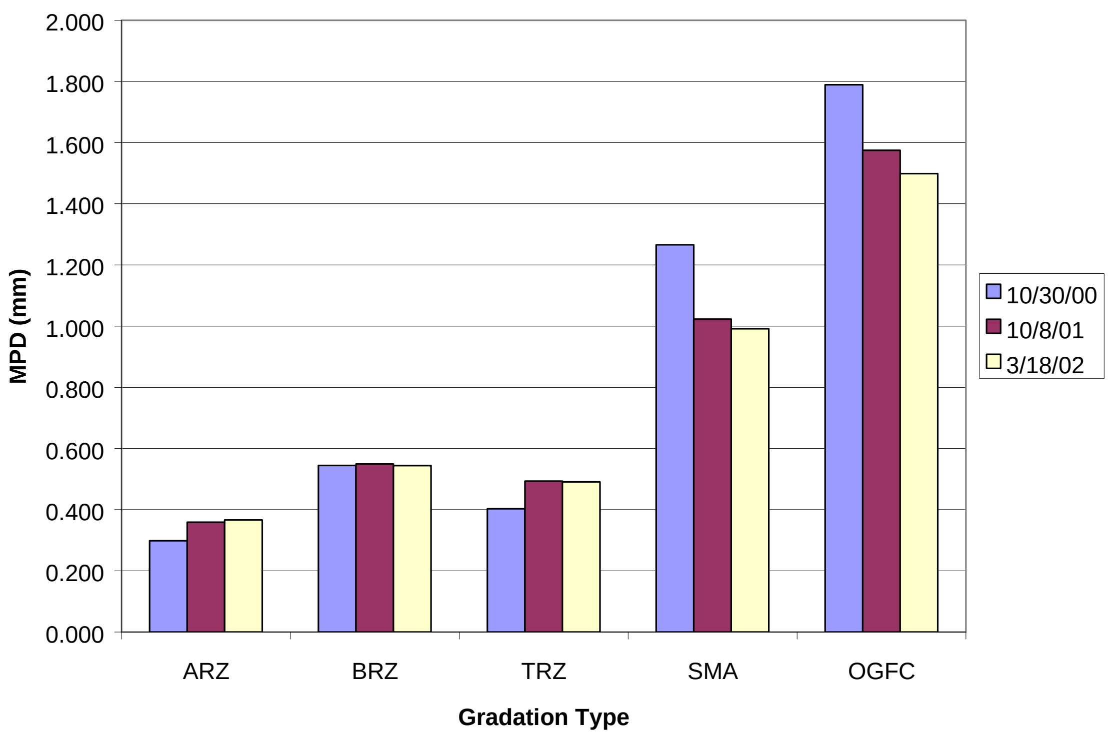

Figure 32. Mean Profile Depth by Gradation Type with Time

Certain aggregate types are prone to polishing under traffic. This action may lead to a reduction in skid resistance with time. Seven major aggregate types were represented at the test track: granite, limestone/slag blend, gravel, Limestone, limestone/RAP blend, quartzite and sandstone. Only three of these aggregate types (granite, gravel and limestone/slag) had numerous replicates. Figure 33 shows the average ribbed tire skid numbers as a function of ESALs for the granite, gravel and limestone/slag sections. Several agencies add slag to limestone blends to improve polish resistance. Figure 33 indicates that the limestone/slag sections had the highest skid number at the time of construction. However, after 4.8 million ESALs, the limestone/slag mixes had lower friction than the granite and gravel mixes.

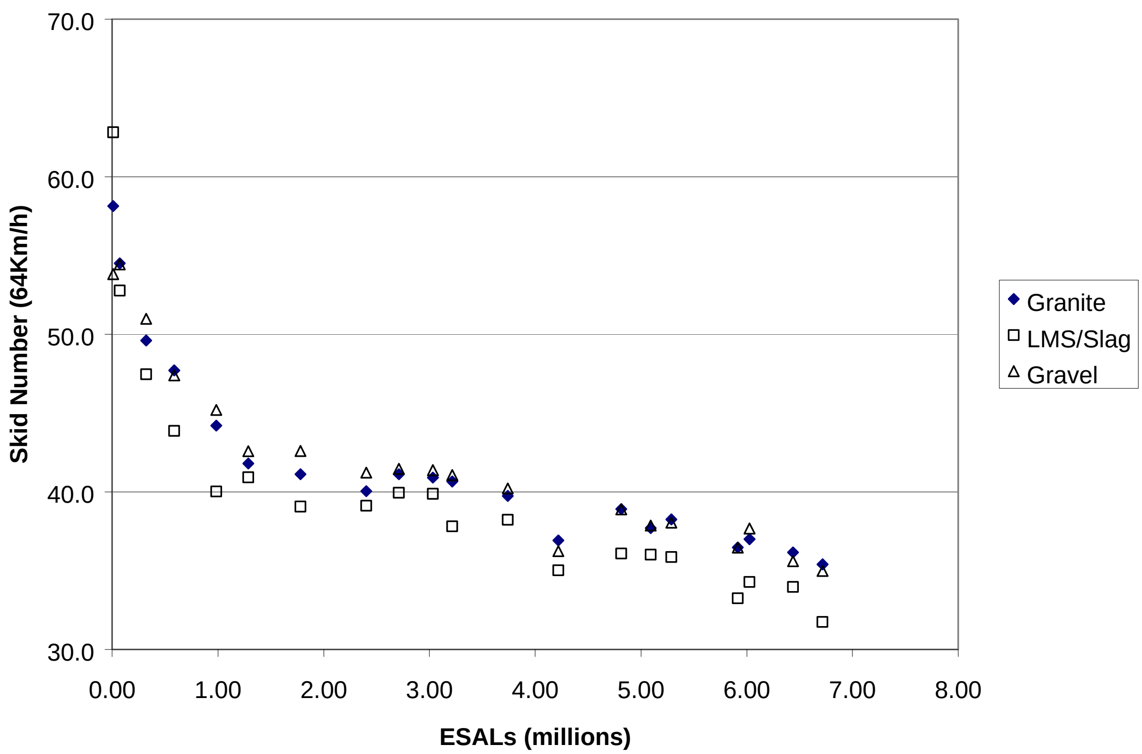

Figure 33. Ribbed Tire Skid Number versus ESALs as a Function of Aggregate Type

The use of hard (low LA Abrasion) cubical aggregates is emphasized for SMA mix designs. Alabama Department of Transportation placed an experimental SMA section on the NCAT test track consisting solely of a native limestone source. The limestone coarse aggregate blend had an LA Abrasion value of 20. The section was placed to evaluate whether the increased macrotexture common to SMA mixes would overcome the tendency of the limestone to polish under traffic. Unfortunately, as shown in Figure 34, the limestone SMA polished causing a reduction in skid resistance. The rate of reduction in skid resistance increased after the application of 5 million ESALs. Once it was clear that the macrotexture of the SMA would not overcome the polish susceptibility of this particular aggregate, a thin maintenance treatment was applied to the section to improve skid resistance. This result does not indicate that limestone cannot be used in SMA, however, one should ensure that the limestone or any other aggregate is resistant to polishing.

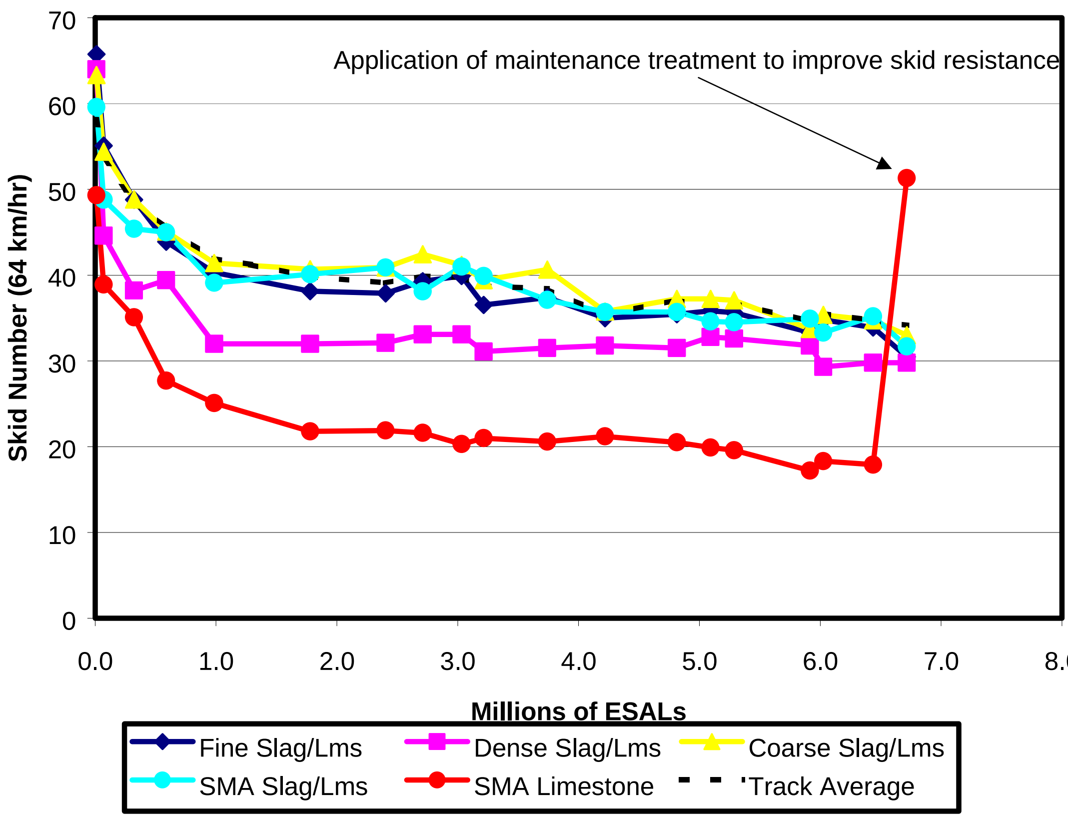

Figure 34. Comparison of Ribbed Tire Skid Number versus ESALs for Track Section Containing Limestone

H. Field Permeability of Track Mixes

In 2001, NCAT representatives conducted a large number of field permeability tests on the Test Track. This testing was in conjunction with a round-robin study for the field permeability device developed at NCAT. A total of eight test sections were tested: E9, N4, N11, N13, S6, S9, S10, and W8. Information on these eight mixes is presented in Table 3.

Of the eight mixes tested, six were designed in accordance with the Superpave mix design system. The remaining two mixes were SMAs. For the six Superpave mixes, four had a gradation passing above the restricted zone, one had a gradation passing below the restricted zone, and one passed through the restricted zone. Average pavement densities for the eight test sections ranged from a low of 92.0 percent Gmm to a high of 94.5 percent Gmm.

Table 3. NCAT Test Track Pavement Sections Utilized in Study

Section	Design	Asphalt Content, %	NMAS, mm	Gradation [1]	Aggregate	Avg. Density, % G_{mm}
E9	Superpave	5.4	12.5	ARZ	Granite	92.9
N4	Superpave	6.8	9.5	ARZ	Limestone/Slag	93.4
N11	Superpave	4.3	12.5	TRZ	Granite	93.1
N13	SMA	6.8	12.5	SMA	Gravel	92.0
S6	Superpave	6.2	12.5	ARZ	Limestone/RAP	92.9
S9	Superpave	4.7	12.5	BRZ	Granite	93.4
S10	Superpave	5.2	12.5	ARZ	Granite	93.7
W8	SMA	7.5	12.5	SMA	Sandstone	94.5

[1] ARZ, TRZ, BRZ ~ Above, Through, and Below the Restricted Zone; SMA ~ Stone Matrix Asphalt

Table 4 presents the average field permeability measurements from the eight test sections. These values are also illustrated in Figure 35. Based on the results shown in Figure 35, all four of the Superpave mixes had gradations passing above the restricted zone and each had relatively low permeability values. The test section containing the Superpave mix with a gradation below the restricted zone had the highest average permeability value. There was a relatively large difference in the permeability characteristics for the two SMA mixes due to the difference in air voids. Table 3 showed that section N13 had the lowest initial density of the eight sections tested and W8 had the highest density.

Table 4. Average Permeability Values

Section	Average Permeability, 10^{-5} cm/sec	No. Observations
E9	23.116	69
N4	32.603	68
N11	48.983	58
N13	55.745	70
S6	34.565	69
S9	79.229	70
S10	1.986	59
W8	3.682	68

All of the mixes tested with the field permeability device had acceptable permeability values. Of the eight, section S9 containing the mix with a gradation passing below the restricted zone had the highest value. All of the mixes having gradations passing above the restricted zone had low field permeability values.

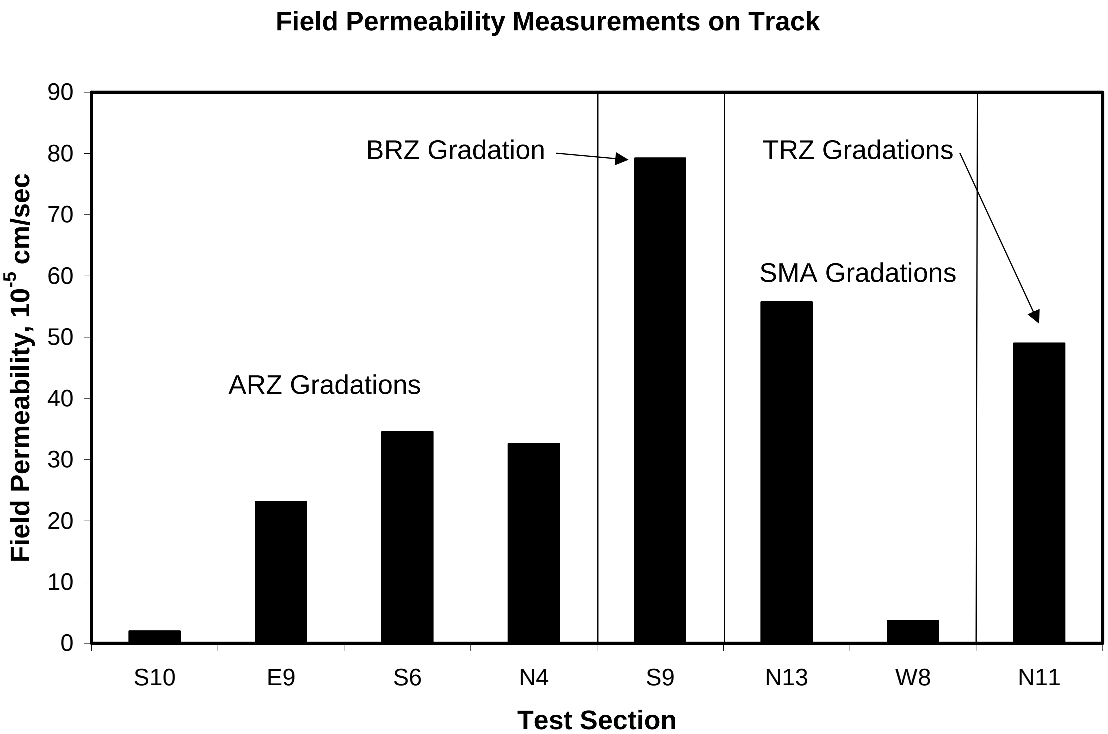

Figure 35: Average Field Permeability Results from Test Track

I. Subsurface Drainage

Pavement drainage at the Test Track was provided with a permeable asphalt treated base (PATB) layer connected to four-inch (100 mm) perforated PVC edge drains. Outlets were spaced at 500 ft (152 m) intervals. A schematic of the pavement and drainage structure is shown in Figure 36.

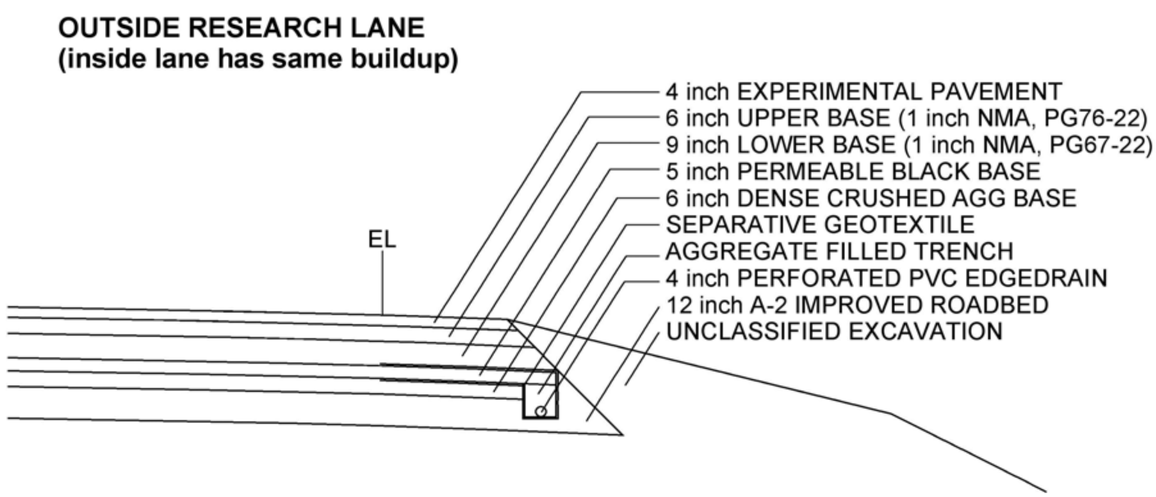

Figure 36. Schematic of Pavement and Drainage Cross Section

The PATB, referred to as permeable black base in Figure 36, was designed according to Alabama Department of Transportation specification section 327E. The PATB consisted of a #67 stone from Columbus Granite mixed with 2.5 percent PG 67-22. The gradation of the PATB is shown in Table 5.

Table 5. PATB Gradation

Sieve Size, in (mm)	Percent Passing
1 ½ (37.5)	100
1 (25.0)	97
¾ (19.0)	87
½ (12.5)	53
3/8 (9.5)	32
#4 (4.75)	7
#8 (2.36)	3
#200 (0.075)	1

Tipping buckets connected to CR10X Data loggers were installed on three outlets adjacent to sections N3, S8 and S1. Both sections N3 and S8 were in fill sections while section S1 was a cut section. Water flow was regularly observed from the outlet pipes during rain events. An example of the outflow is shown in Figure 37. From Figure 37, it is evident that the outlet pipe adjacent to section N3 had the greatest outflow with a peak flow of 0.85 cubic feet per minute. Peak flows for section S1 and S8 were approximately 0.025 cubic feet per minute. This difference is typical of the data and somewhat surprising considering section N3 and its adjacent sections were fine graded ARZ mixes. One possible explanation is that more water entered the joint between the driving lane and shoulder due to the relatively impermeable surface. However, the moisture is more likely to be coming into the drainage system from the embankment outside the shoulder. This is consistent with the short time delay between the peak rainfall and peak outflow from section N3.

J. Subgrade Moisture Content

The moisture in the improved subgrade was controlled to be near optimum (approximately 10%) during compaction. Moisture sensors were installed in November 1999 and were monitored manually until November 2000. At that time the instrumentation was connected to equipment that provided continuous readout. A sample of the data is illustrated in Figures 38-41. For illustration purposes, two sections were selected at random on each of the four sides of the track. Three sections were taken in fill areas, three in cut areas, and two in transition areas. The first manual reading was obtained in November 1999 and found to be approximately 10%. The next manual reading was taken in March 2000 and at that time all of the readings at the 8 locations shown were approximately 22-25%. Each sensor was located in between two different sections therefore the location of each sensor was identified by two sections. After the continuous readings began in November 2000, it appears that the moisture continued to stay at approximately 22-25%.

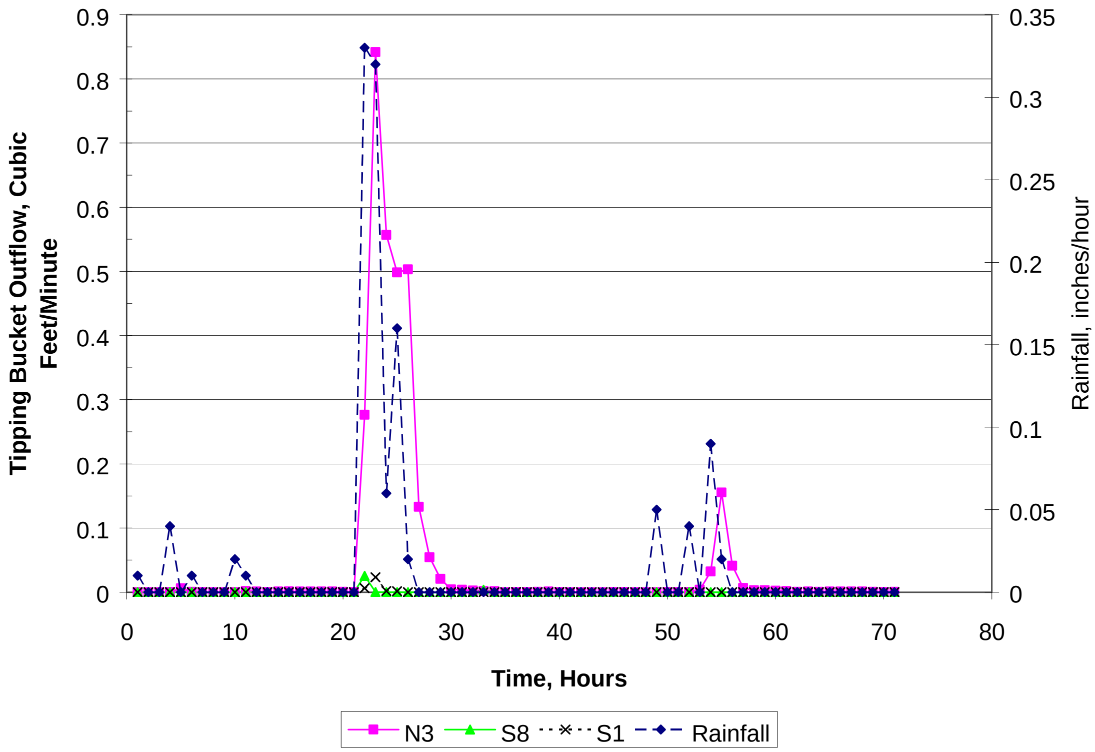

Figure 37. Example Edge Drain Outflows

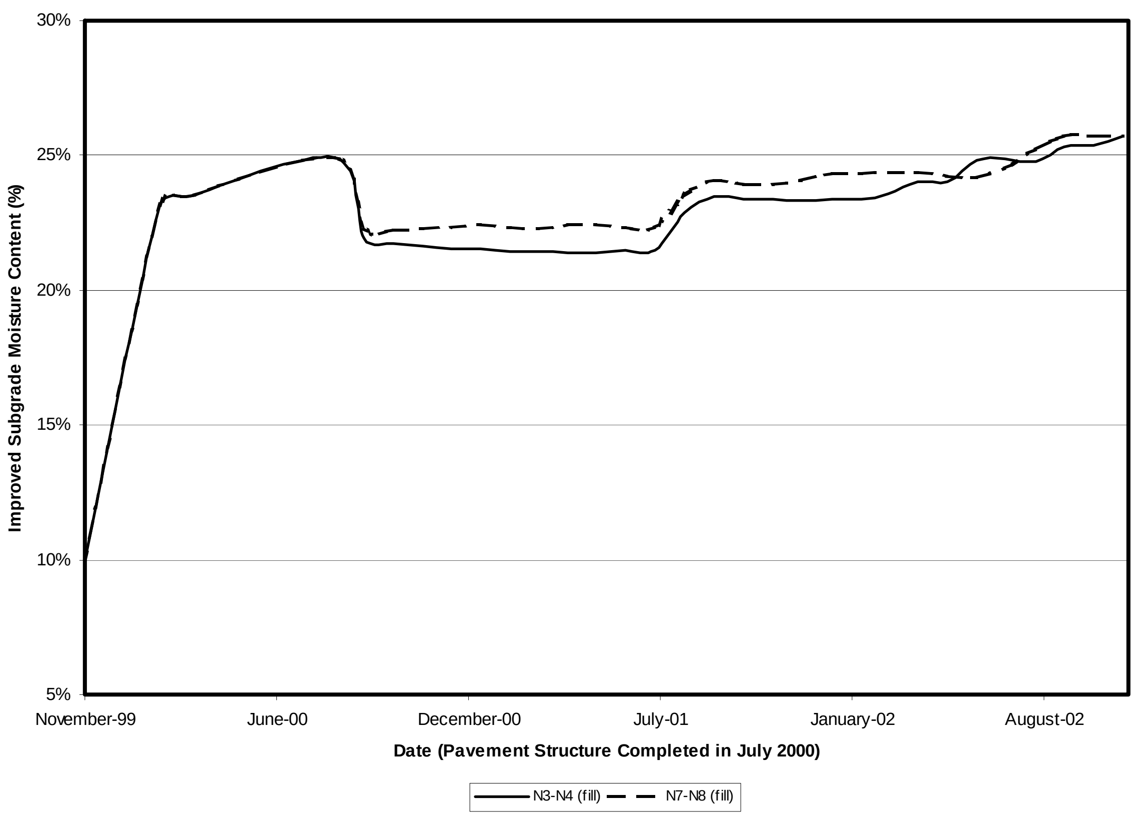

Figure 38. Subgrade Moisture for Section N3-N4 and N7-N8

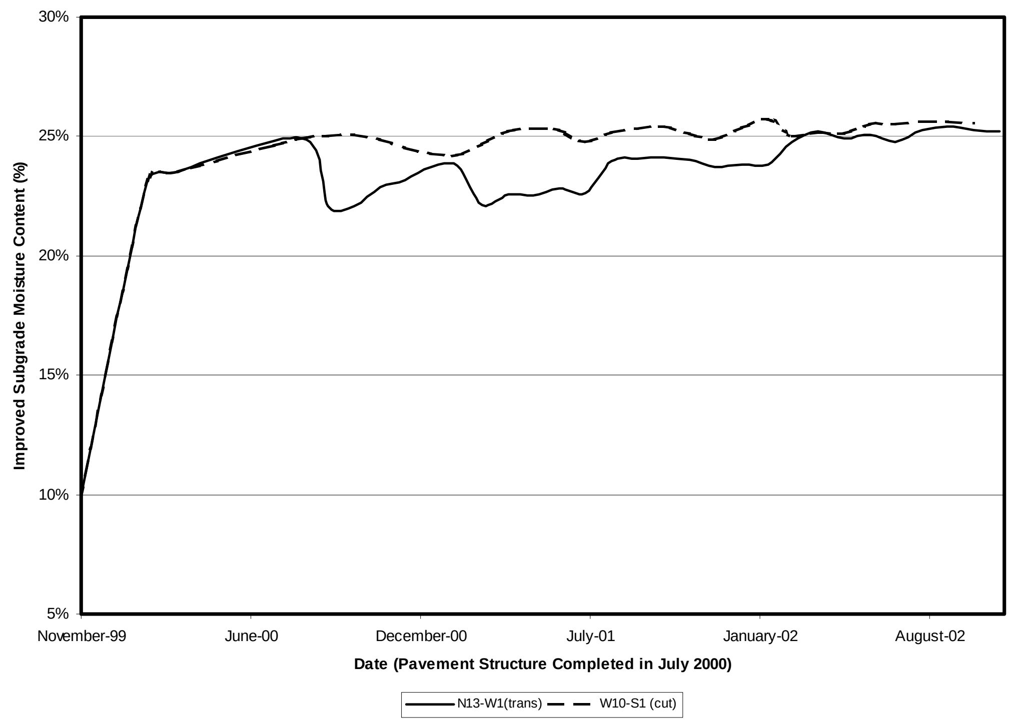

Figure 39. Subgrade Moisture for Sections N13-W1 and W10-S1

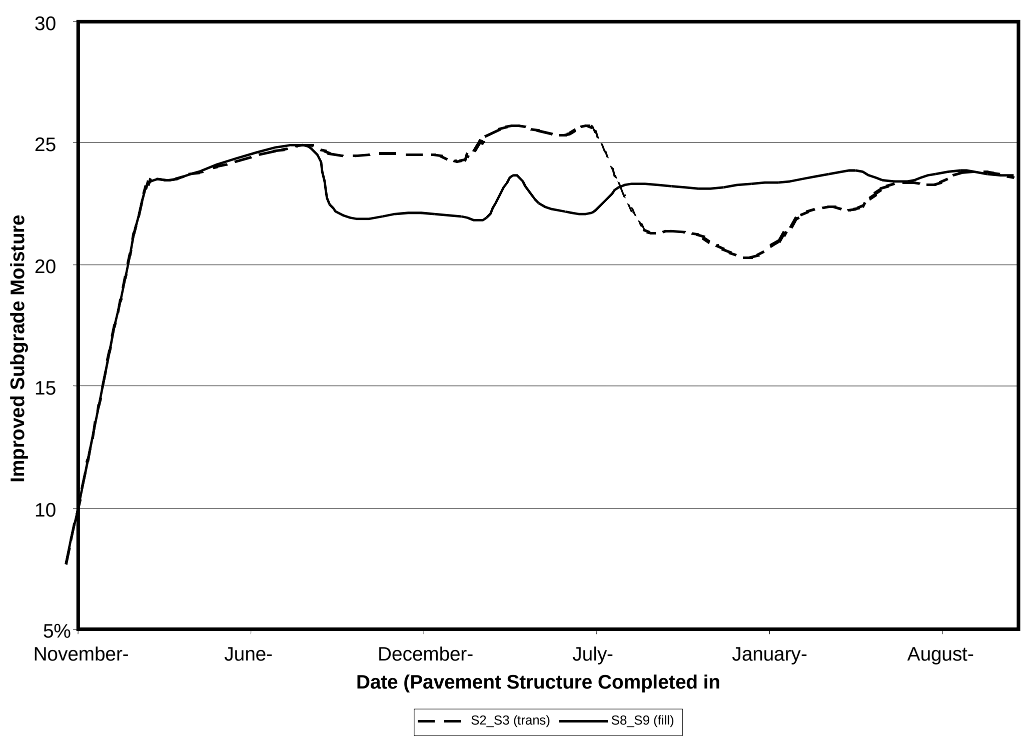

Figure 40. Subgrade Moisture for Sections S2-S3 and S8-S9

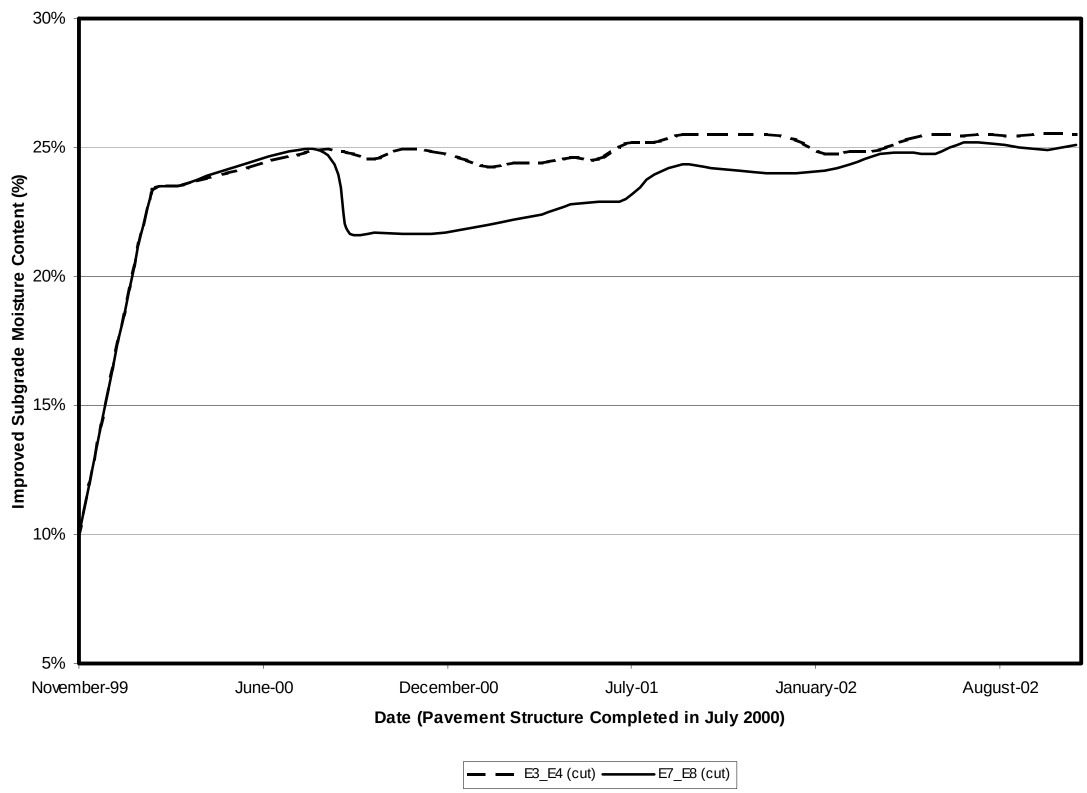

Figure 41. Subgrade Moisture for Sections E3-E4 and E7-E8

This consistent moisture content is interesting since much of the track was built on fill areas and much was built in cut areas. There appears to be no difference between cut and fill areas. Also the amount of rainfall varied significantly during various parts of the year. Regardless of the changes in climate or whether or not it was in a fill or cut area, the moisture content stayed approximately constant after it initially reached the 22-25% range. Of course the sampling location was well below the surface so any variation in moisture would be minimized. In pavement design it is typically assumed that the subgrade will become saturated with time underneath the watertight pavement surface. The data collected at the track tends to support this assumption.

K. Mixture Performance

The primary initial purpose of the track was to evaluate various mixture types and to evaluate the ability of laboratory tests to predict performance. As stated earlier the only expected distress at the Track was some type of surface related problem such as rutting. The pavement was designed strong enough to prevent fatigue cracking and due to the relative short time of evaluation durability problems were not expected.

All of the sections performed very well for the first 9 million ESALs. In fact no maintenance was required on any section other than one section in one of the curves that had a friction problem. The mixture in this section used a limestone aggregate that was known to polish. It was selected for use in an SMA to determine if the coarse surface texture would continue to provide good skid resistance as it was subjected to traffic. The friction was monitored on a regular basis and when the friction fell below an acceptable point, the test section was immediately overlaid with a maintenance course to improve friction.

The measured rutting for each of the tangent sections is shown in Figure 42 and Table 6. First, it is important to notice that the level of rutting is very small. The worst section N3 only had about 6mm (0.25 inches) of rutting. The scale on the figure goes up to approximately 12.5mm (0.5 inches). This is the level (0.5 inches) that most state DOTs begin to consider rutting to be significant, but as clearly shown in the figure all of the sections had rutting values well below 12.5mm (0.5 inches).

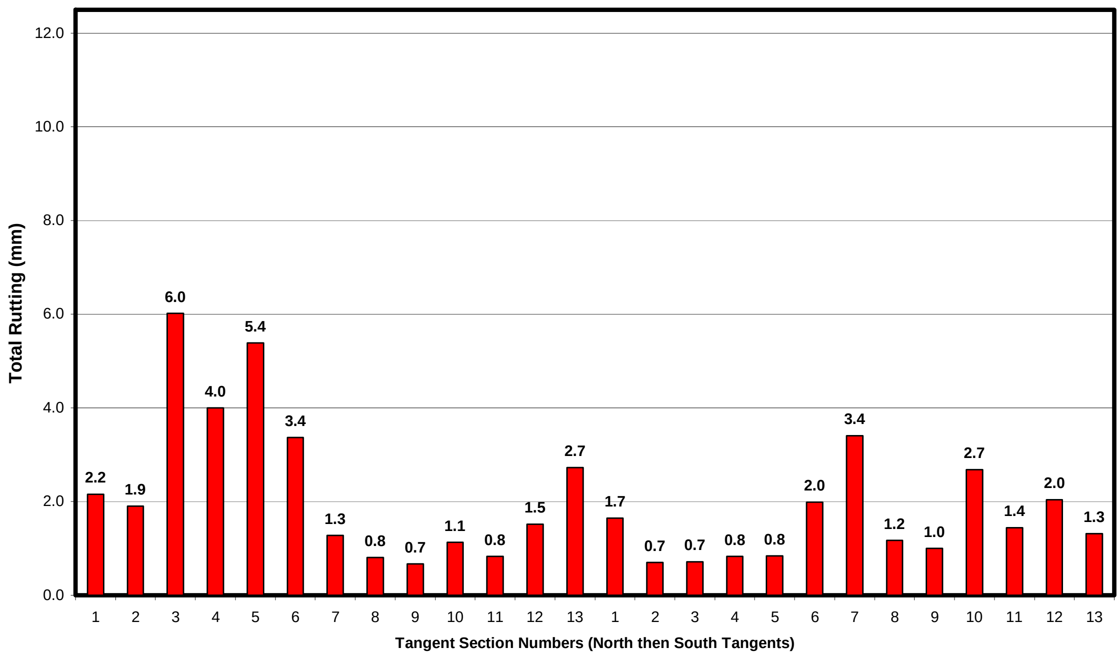

Figure 42. Bar Chart Showing Rut Depths

Table 6. Rut Depth After 9 Million ESALS (measured with Dipstick)

		Rutting (in)			Overall Average (in.)
Test Section	**Location**	**Lt**	**Rt**	**Avg**	
N 1	1	0.05	0.15	0.10	
N 1	2	0.04	0.10	0.07	
N 1	3	0.07	0.10	0.08	**0.08**
N 2	1	0.04	0.14	0.09	
N 2	2	0.02	0.10	0.06	
N 2	3	0.03	0.11	0.07	**0.07**
N 3	1	0.15	0.31	0.23	
N 3	2	0.15	0.29	0.22	
N 3	3	0.22	0.30	0.26	**0.24**
N 4	1	0.15	0.14	0.14	
N 4	2	0.15	0.18	0.16	
N 4	3	0.18	0.15	0.16	**0.16**
N 5	1	0.14	0.26	0.20	
N 5	2	0.13	0.26	0.20	
N 5	3	0.19	0.28	0.24	**0.21**
N 6	1	0.14	0.15	0.15	
N 6	2	0.07	0.19	0.13	
N 6	3	0.04	0.20	0.12	**0.13**
N 7	1	0.04	0.05	0.05	
N 7	2	0.10	0.05	0.08	
N 7	3	0.02	0.03	0.03	**0.05**
N 8	1	0.00	0.05	0.03	
N 8	2	0.01	0.05	0.03	
N 8	3	0.02	0.06	0.04	**0.03**
N 9	1	0.01	0.04	0.03	
N 9	2	0.01	0.03	0.02	
N 9	3	0.01	0.05	0.03	**0.02**
N 10	1	0.02	0.06	0.04	
N 10	2	0.01	0.08	0.05	
N 10	3	0.02	0.07	0.04	**0.04**
N 11	1	0.02	0.04	0.03	
N 11	2	0.02	0.06	0.04	
N 11	3	0.03	0.02	0.02	**0.03**
N 12	1	0.04	0.05	0.04	
N 12	2	0.06	0.08	0.07	
N 12	3	0.06	0.07	0.06	**0.06**
N 13	1	0.07	0.14	0.11	
N 13	2	0.08	0.14	0.11	
N 13	3	0.08	0.13	0.10	**0.11**

Table 6 (cont.) Rut Depth After 9 Million ESALS

Test Section	Location	Rutting (in) Lt	Rt	Avg	Overall Average (in.)
S 1	1	0.09	0.08	0.09	
S 1	2	0.05	0.09	0.07	
S 1	3	0.03	0.05	0.04	**0.06**
S 2	1	0.01	0.04	0.02	
S 2	2	0.02	0.04	0.03	
S 2	3	0.02	0.04	0.03	**0.03**
S 3	1	0.01	0.05	0.03	
S 3	2	0.02	0.04	0.03	
S 3	3	0.04	0.02	0.03	**0.03**
S 4	1	0.01	0.04	0.02	
S 4	2	0.04	0.05	0.05	
S 4	3	0.01	0.04	0.03	**0.03**
S 5	1	0.01	0.04	0.03	
S 5	2	0.01	0.06	0.04	
S 5	3	0.02	0.05	0.04	**0.03**
S 6	1	0.06	0.10	0.08	
S 6	2	0.07	0.11	0.09	
S 6	3	0.03	0.10	0.06	**0.08**
S 7	1	0.11	0.15	0.13	
S 7	2	0.11	0.18	0.15	
S 7	3	0.11	0.15	0.13	**0.14**
S 8	1	0.04	0.06	0.05	
S 8	2	0.02	0.06	0.04	
S 8	3	0.04	0.06	0.05	**0.05**
S 9	1	0.01	0.08	0.04	
S 9	2	0.00	0.07	0.04	
S 9	3	0.02	0.06	0.04	**0.04**
S 10	1	0.05	0.09	0.07	
S 10	2	0.14	0.11	0.13	
S 10	3	0.11	0.13	0.12	**0.1**
S 11	1	0.03	0.04	0.03	
S 11	2	0.13	0.04	0.09	
S 11	3	0.05	0.05	0.05	**0.06**
S 12	1	0.08	0.06	0.07	
S 12	2	0.11	0.05	0.08	
S 12	3	0.11	0.07	0.09	**0.08**
S 13	1	0.03	0.03	0.03	
S 13	2	0.07	0.05	0.06	
S 13	3	0.08	0.05	0.06	**0.05**

The data in Table 6 shows that generally the rutting is higher in the right wheel path than in the left path. There are probably at least 2 reasons for this. First of all, there is a 2 percent transverse slope. This slope resulted in a slightly heavier load on the right side than on the left side. Secondly, the material adjacent to the slope, likely does not provide as much confinement as the left lane in the roadway. Hence, more rutting would be expected in the right wheel path.

Unless noted otherwise, the rutting provided in this report was determined with a dipstick. There are two ways that this was done, 3-point deformation and 6-point deformation. A comparison of the 2 procedures using the dipstick is provided in Figure 43. Notice that there is very little difference in the 2 methods.

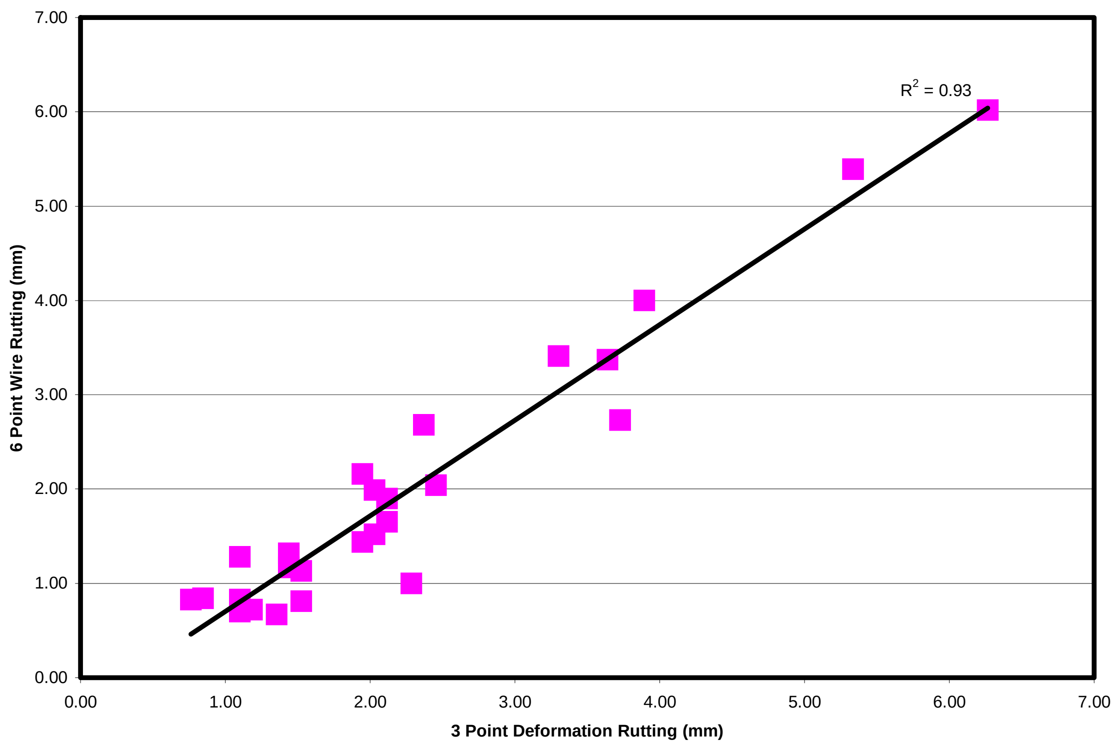

Figure 43: Comparison of 3-point Deformation and 6-Point Deformation

It is also interesting to note that the variability of the rutting within a section is very low. Even though the overall rutting is low, it appears to be consistent between tests within each section.

Some of the sections appear to have significantly higher rutting values than others. Remember that some of the sections were designed so that they would be more likely to rut. For example the sections with the most rutting were N3 and N5. Both of these sections were designed with 0.5% additional asphalt binder so that they might experience rutting when exposed to this high level of traffic. Also some of the sections used aggregates that were marginal and it was expected that this could cause some rutting problems.

Another item of interest is the observed rutting rates. Figure 44 shows the average rutting of all of the sections. It indicates that the rutting rate was relatively high during the first 800,000 ESALs even though much of the traffic was applied during the winter months. It appears that this first significant rate of rutting was caused more by initial seating of the aggregate and initial compaction. For example if the average densification in the top 100mm (4 inches) increased by 1%, this should result in an average rut depth of 1mm.

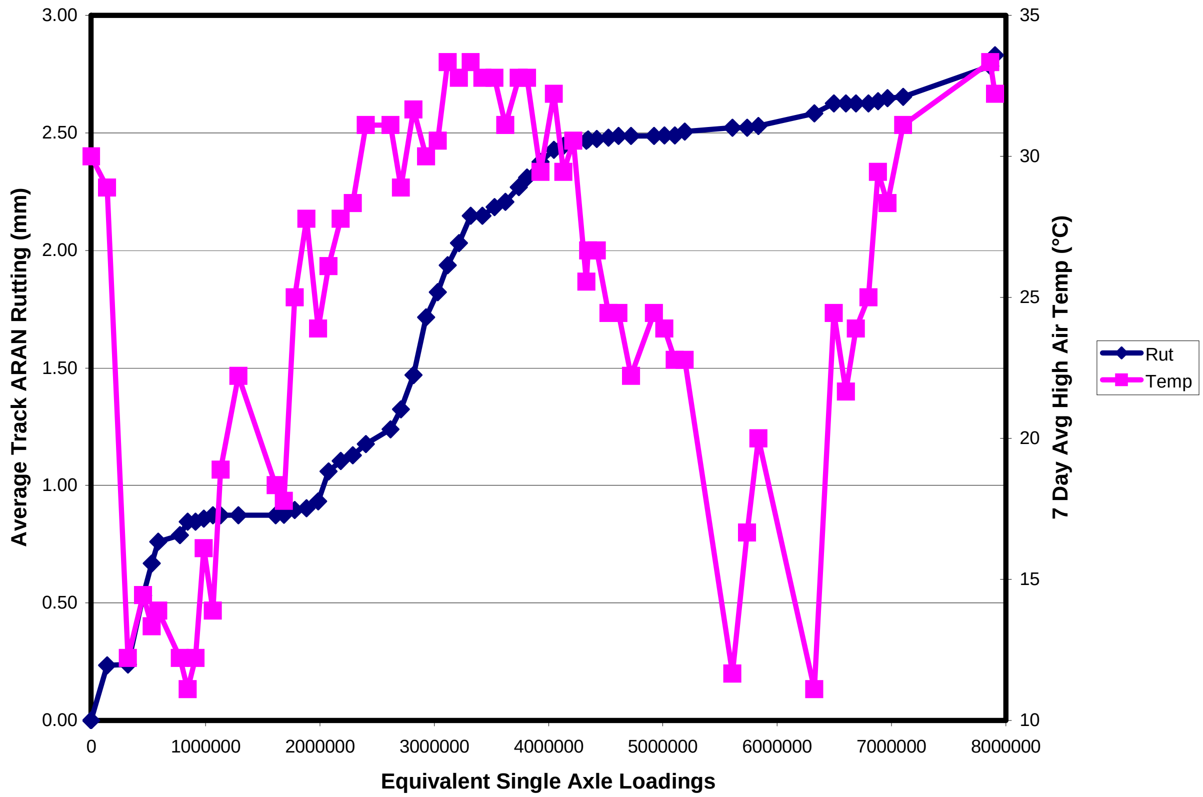

Figure 44. Effect of Air Temperature on Rutting Rate

After this initial densification and seating of the aggregate the rutting rate was reduced to near zero until the average 7 day maximum daily temperature reached approximately 28°C at which time the rutting rate again began to climb. However, the rate appeared to be a little less than the initial rate. Notice that after the 7 day maximum daily temperature dropped below approximately 28°C the rate of rutting almost went to zero again. The rate of rutting stayed near zero until the temperature exceeded approximately 18 C at which time the rate began to increase a little. These higher temperatures represent the second summer of traffic. The rate was much lower during this second summer than it was for the two earlier rate increases. Based on this observation it appears that a mix that is properly designed will stabilize within a couple of years due to aging and compaction. The data also shows that the initial seating and densification resulted in an overall average rut depth of approximately 0.8mm. The first summer resulted in an additional 1.7mm of rutting and the second summer resulted in an additional 0.5mm of rutting on average.

One mini experiment involving 10 sections was set up to look at the effect of PG grade, asphalt content, and fine graded vs. coarse graded mixes. These sections were identified as N1 through N10. A plot of the results is provided in Figure 45. One observation from this plot is that the mixes with modified asphalts (PG-76) had significantly lower rutting (66% lower). This indicates the importance of bumping the PG grade on high volume roads as specified by Superpave. Another observation is that the SBS and SBR gave very similar results.

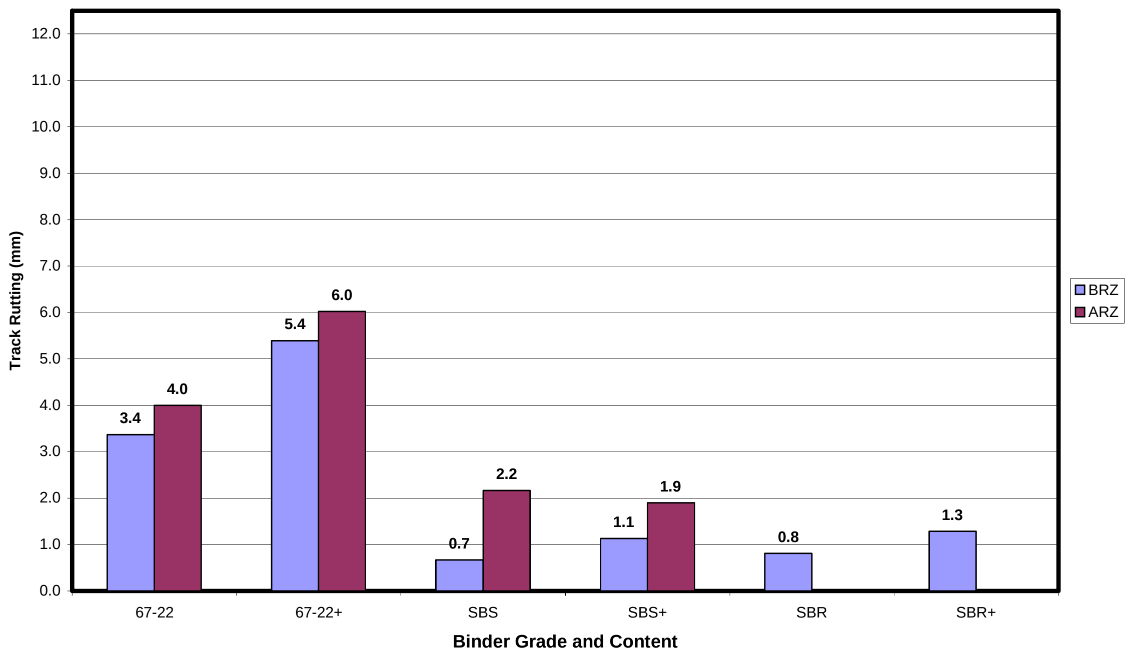

Figure 45. Comparison of Rutting to AC Grade, Binder Content, and Fine vs. Coarse Graded Mixes

Increasing the asphalt content by 0.5% resulted in an increase of 54% in the rutting of the unmodified mixes. When the mixes were modified the increase in rutting as a result of the increased asphalt content was very small (less than 1 mm). This indicates that one may be able to use slightly higher asphalt content with modified asphalts to improve durability without causing a loss in performance due to rutting.

There were 3 mini experiments to look at fine graded vs. coarse graded mixes (Figure 46). The section numbers for these experiments were N4, N6, S6, S7, S9, and S10. The data clearly shows that there was very little difference in the amount of rutting of fine graded and coarse graded mixes. Hence, from a rutting standpoint, good performance can be obtained with fine graded as well as with coarse graded mixes.

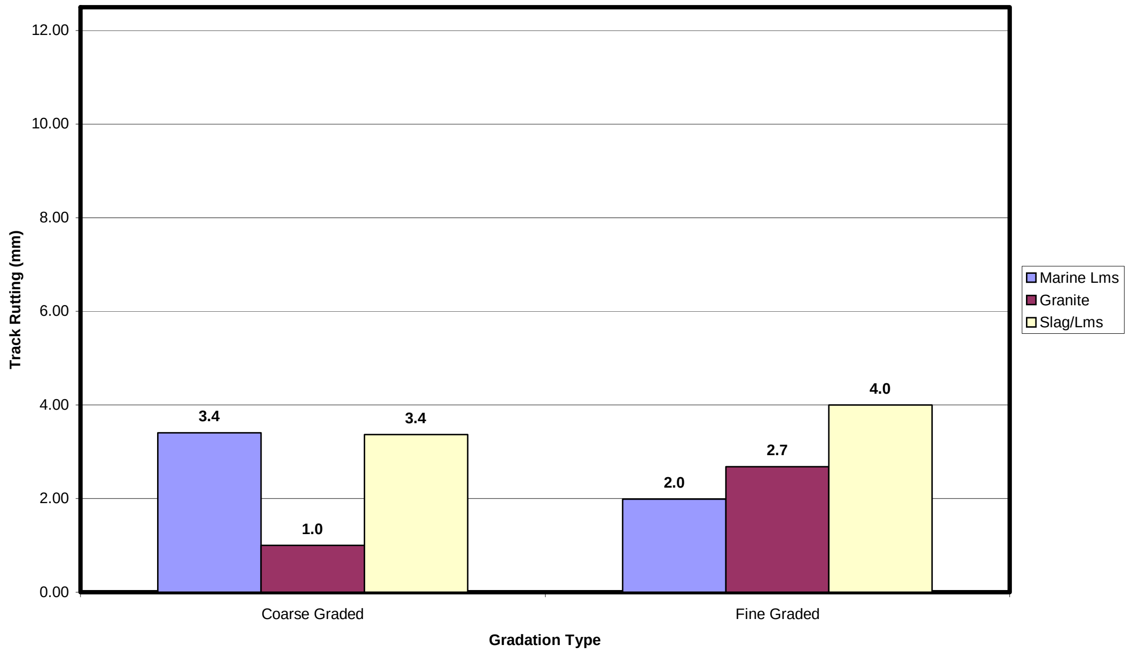

Figure 46. Comparison of Fine Graded vs. Coarse Graded Mixes

Of course one of the keys to ensuring good performance is to have a test that is accurately related to performance. Several tests have been used over the years to predict performance and new tests are being evaluated. Several laboratory tests were evaluated for the purpose of predicting performance. A lot of details about the tests are not provided here but a plot to show the trend of each of these tests with performance is provided. The tests that were evaluated included: wheel tracking tests, Superpave simple shear, dynamic modulus, creep, confined repeated load test, and gyratory shear tests. Keep in mind that the rutting observed at the track was very small so it is difficult for these tests to accurately predict the rutting. If the rutting numbers were higher then a better evaluation could be made.

The results of tests from the asphalt pavement analyzer are provided in Figure 47. The Hamburg results are shown in Figure 48. The rotary LWT rut testing results are shown in Figure 49. The gyratory testing machine (GTM) strain results are provided in Figure 50. The gyratory shear ratio results are provided in Figure 51. The confined repeated load test results are provided in Figure 52. The dynamic modulus results are provided in Figure 53. The SST-repeated shear, constant height results are provided in Figure 54.

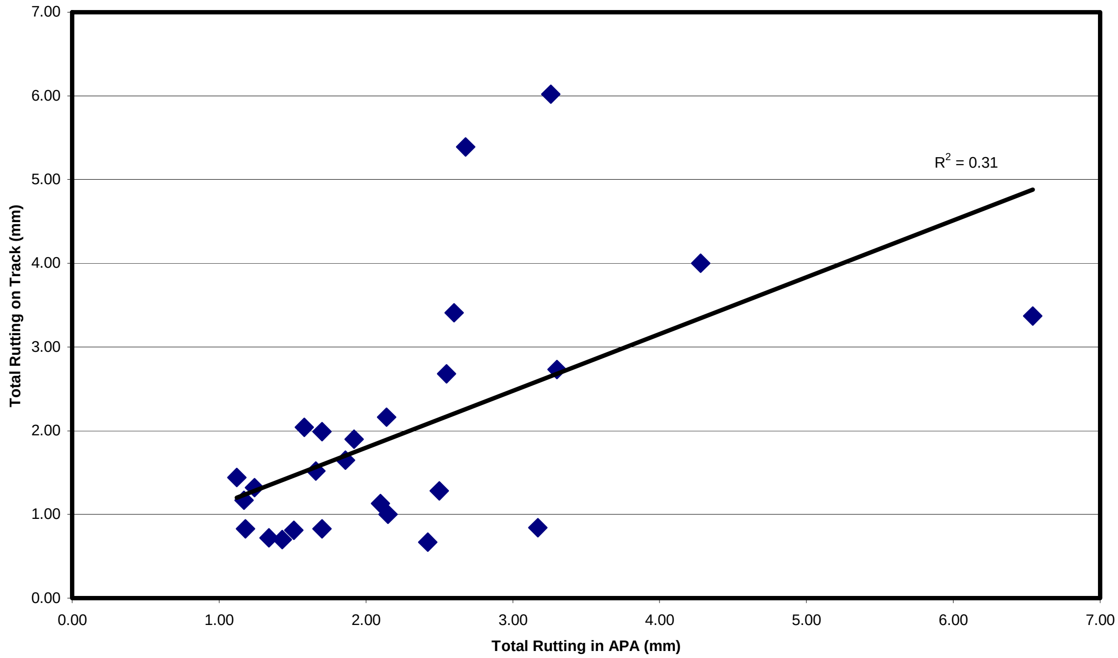

Figure 47. Test Track Rutting vs. APA

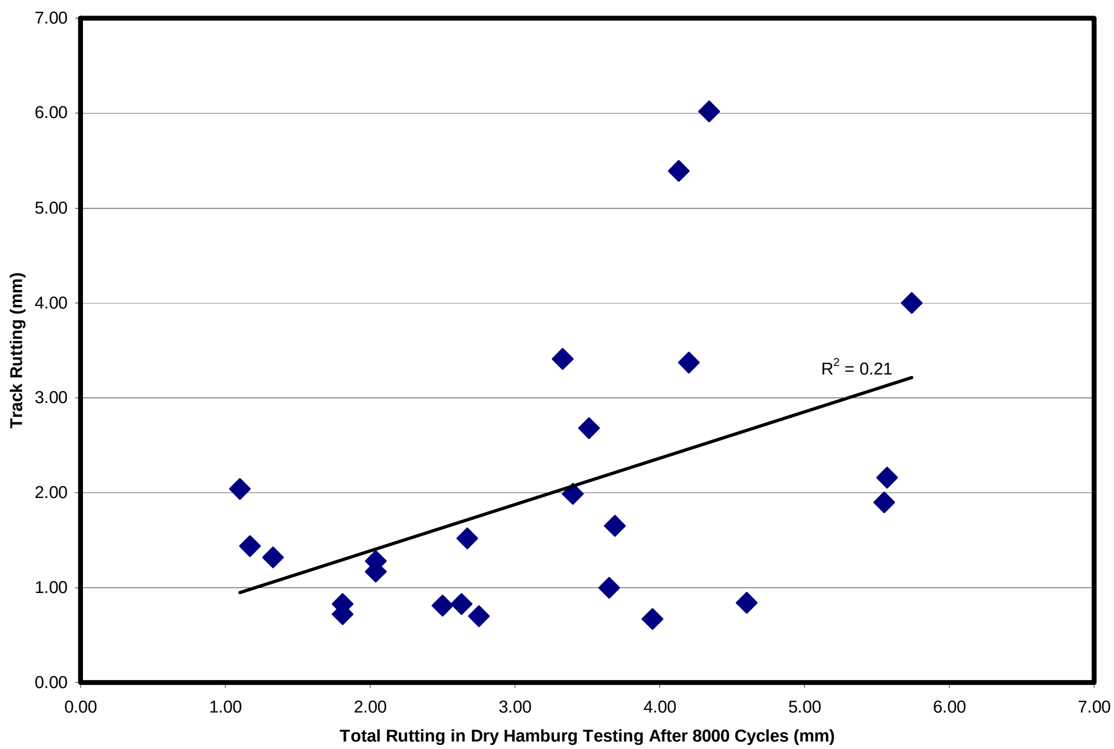

Figure 48. Test Track Rutting vs. Hamburg Wheel Tracking Test

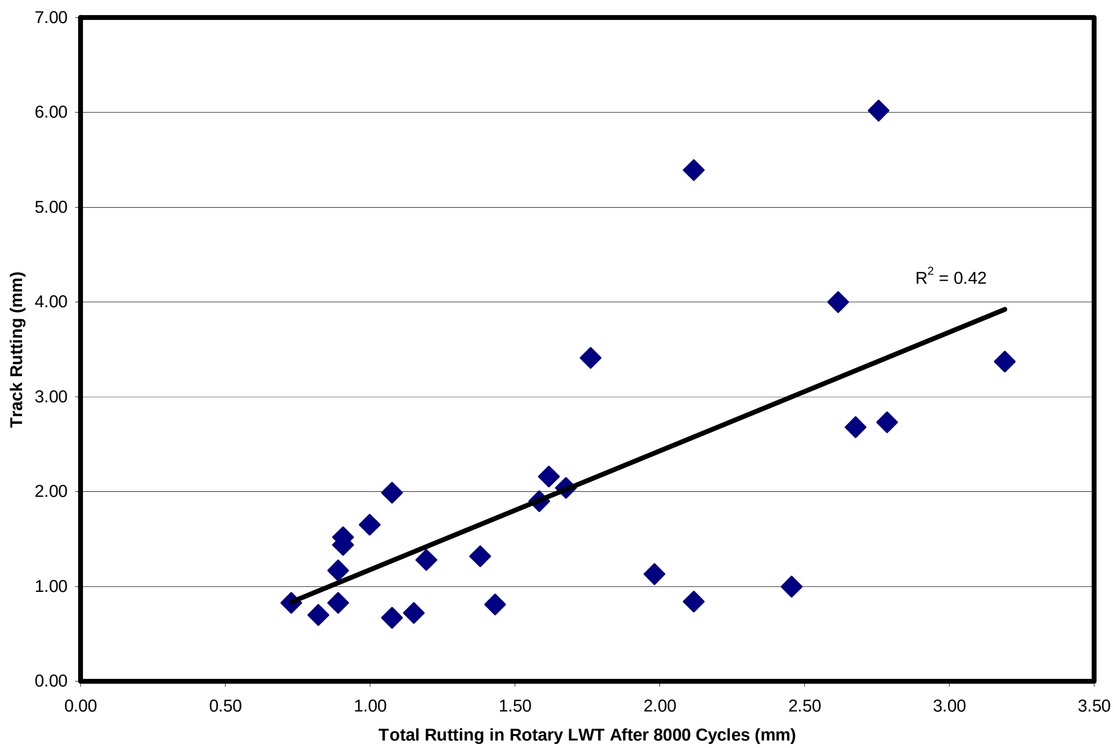

Figure 49. Test Track Rutting vs. Rotary LWT Rut Tester

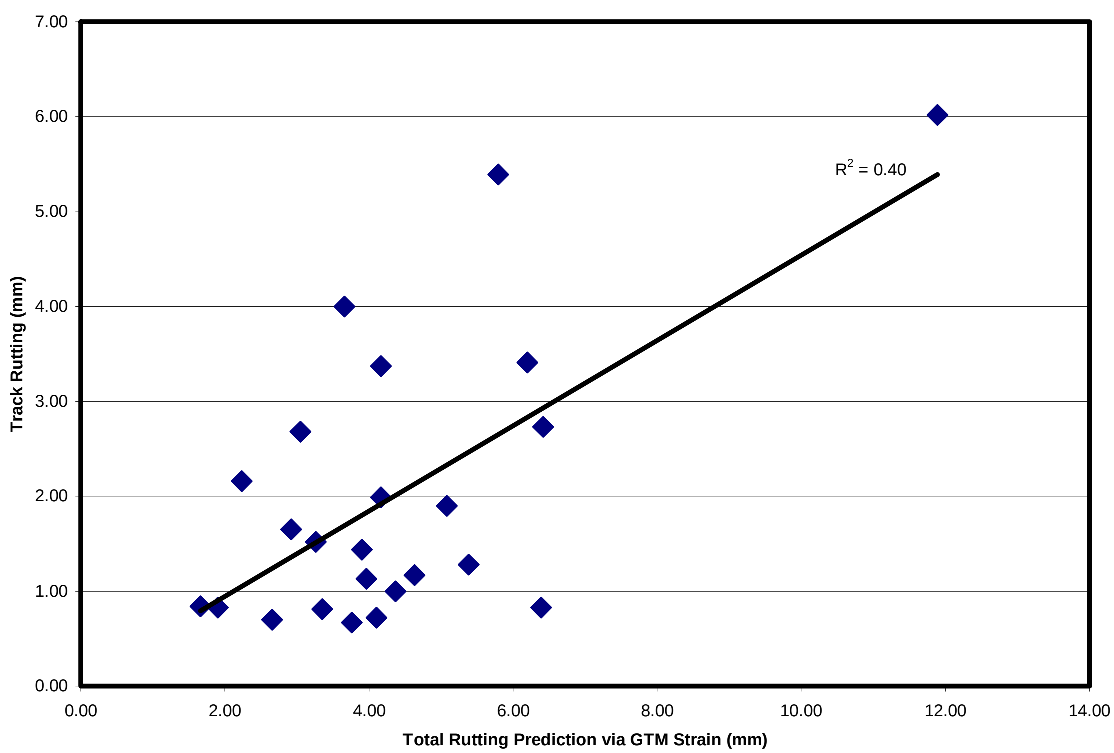

Figure 50. Test Track Rutting vs. GTM Strain

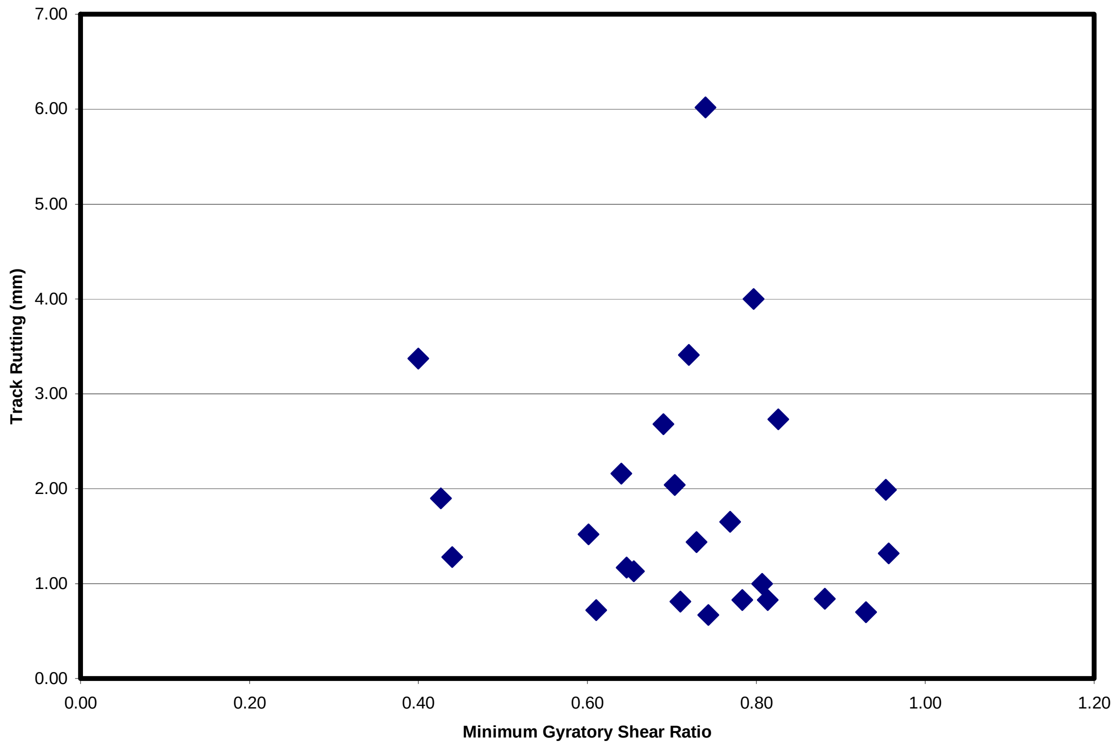

Figure 51. Test Track Rutting vs. Gyratory Shear

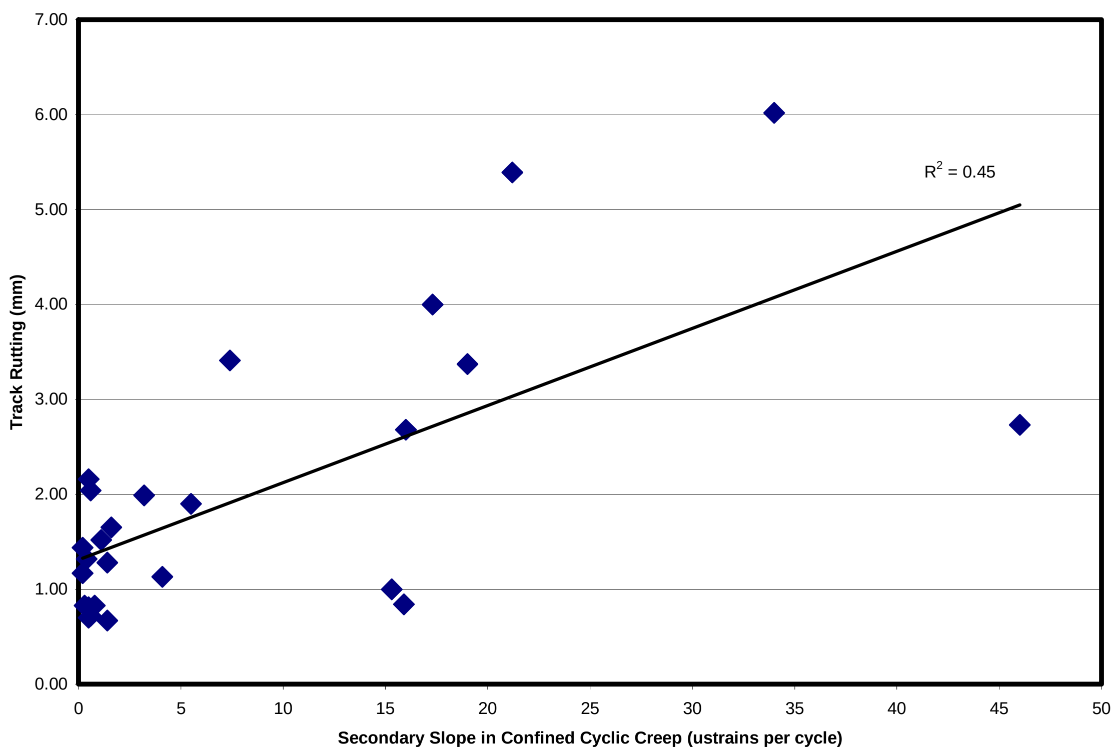

Figure 52. Test Track Rutting vs. Confined Repeated Load Test

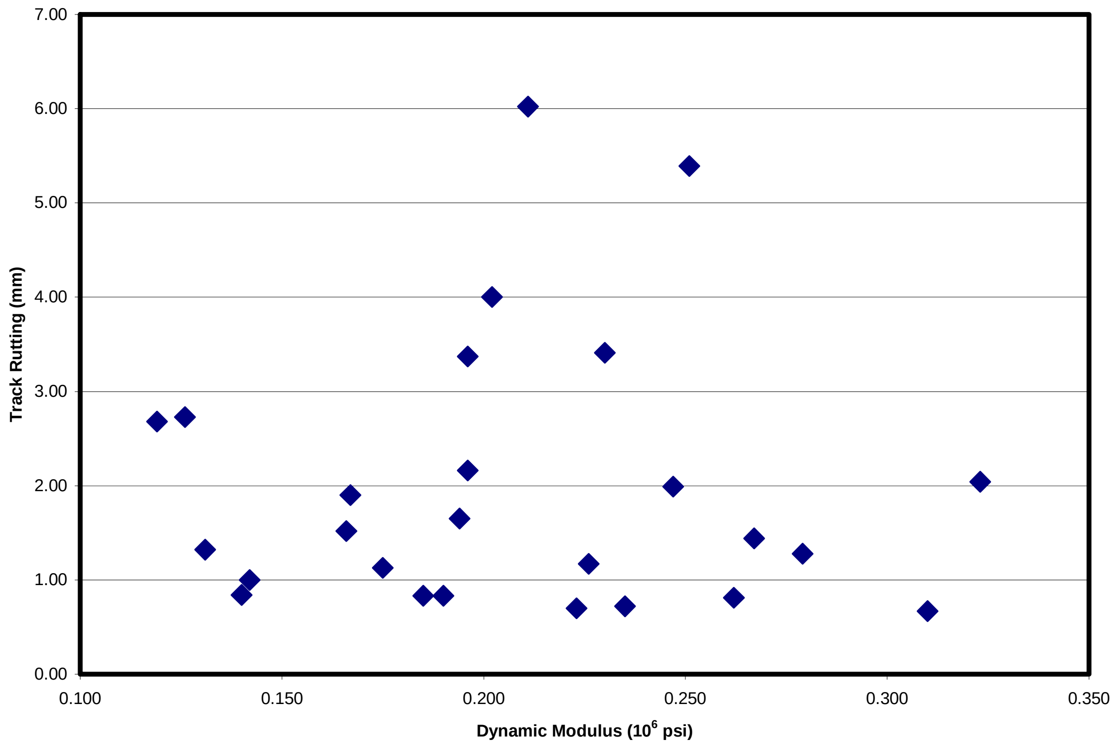

Figure 53. Test Track Rutting vs. Dynamic Modulus

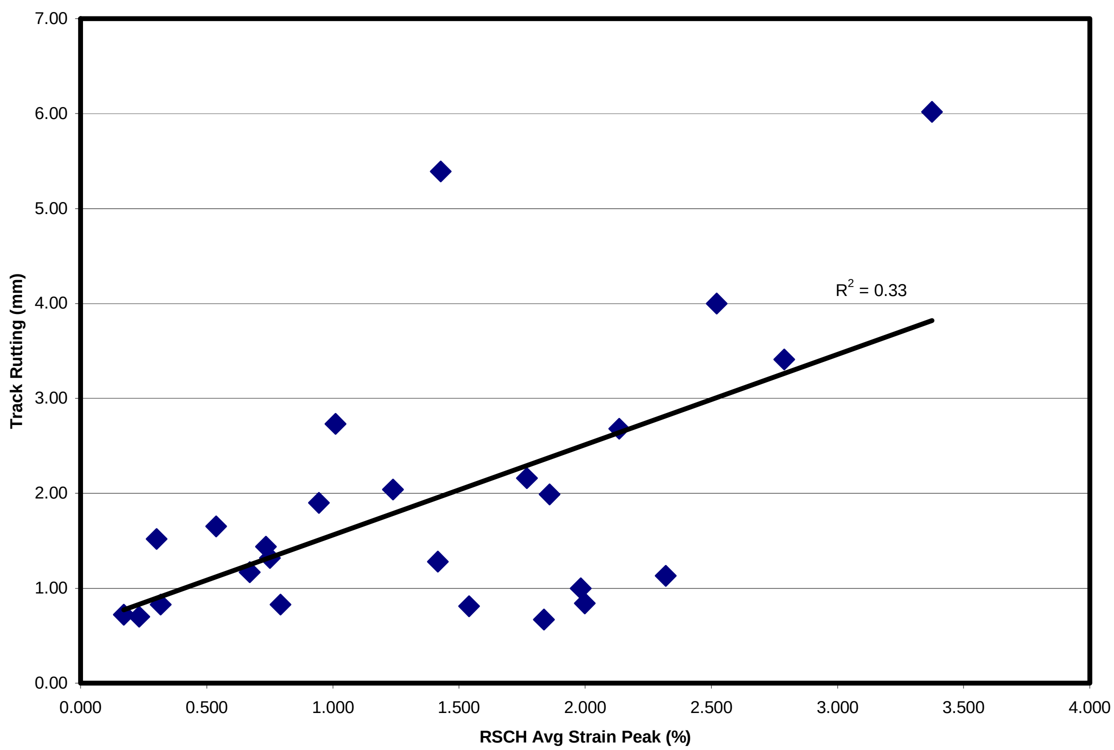

Figure 54. Test Track Rutting vs. SST Test-Repeated Shear, Constant Height

The rutting results are very preliminary; hence, one must be careful in making too many conclusions. However, it is worth noting that there appears to be no relation between rutting and dynamic modulus. There is a reasonable relationship with the confined repeated load test. The rut testers do show a trend with performance. More analysis is needed and will be performed at the completion of traffic.

L. Pavement/Tire Noise Study

Introduction

The FHWA noise abatement criteria states that noise abatement must be considered for residential areas when the traffic noise levels approach or exceed 67 dB. To accomplish this many areas in the United States are building large sound barrier walls at a cost of one to five million dollars per roadway mile. In January of 2002 the National Center for Asphalt Technology initiated a research study with the objective to develop safe, quiet and durable asphalt pavement surfaces. The first step towards accomplishing this objective is to develop a fast and scientifically reliable method for measuring the acoustical characteristics of pavement surfaces.

Measurement of Road Noise

Two general methods have been developed for measuring pavement noise levels in the field: the statistical by-pass approach as defined by ISO Standard 11819-1 and the close proximity method (CPX) as defined by ISO Standard 11819-2.

Statistical By-Pass Method. It consists of placing microphones at a defined distance from the vehicle path at the side of the roadway. It calls for placing microphones 25 feet from the center of the vehicle lane and at a height of 4 feet above the pavement and requires that the noise characteristics and speed of 180 vehicles be obtained (100 automobiles and 80 dual-axle and multi-axle trucks).

Close-Proximity Method (CPX). This method consists of placing microphones near the tire/pavement interface to directly measure the tire/pavement noise levels. In the close-proximity method the microphones are mounted as shown in Figure 55. They are mounted inside an acoustical chamber (each side of the chamber is covered with acoustical sound deadening material). The purpose of this is to eliminate the noise from traffic while testing.

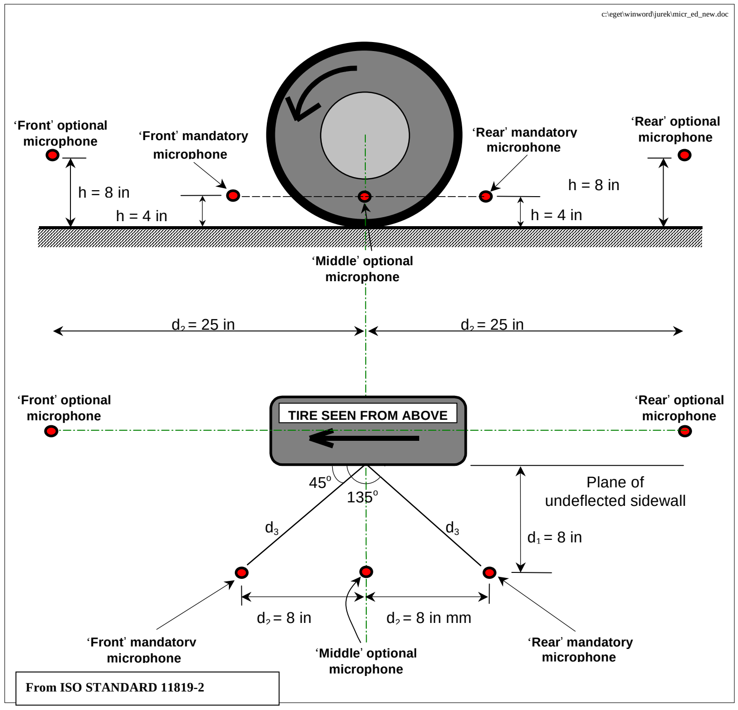

Figure 55. Microphone Layout for Close-Proximity Trailer

The National Center for Asphalt Technology designed and built a CPX trailer for the Arizona Department of Transportation (ADOT) during the Fall of 2001. It was delivered to ADOT in late January 2002 and is being used by ADOT to evaluate a number of pavement surfaces in Arizona. In October 2002 the second generation CPX trailer was delivered to NCAT and is now being used to conduct tudies at the Track. Figure 56 shows a picture of that new trailer.

Figure 56. NCAT CPX Trailer

The first goal of the study is to finalize the development of the test procedure. A decision needs to be made as to the type of tire that will be used for the conduct of the test. Eight tires will be used on the trailer with the Track as the test surface. The following preliminary results (Table 7) provide an indication of how important this issue is. A change of 3 dB in the noise level is significant – the human ear will differentiate this change in noise level.

Table 7. Preliminary Results from NCAT Test Track Comparing Tire Type

Tire	Noise level (dB)
UniRoyal	93.5
Firestone	94.9
MasterCraft	96.9

Also, the track data will be used to evaluate the effect of surface type on noise levels. Based on this work it may be possible to determine the relationship between surface texture and noise. It is anticipated that the testing on the track surfaces will be completed in January 2003 and the results published in February.

IX. OBSERVATIONS TO DATE

At the time this report was written approximately 9.4 million ESALs had been applied to the track. The remaining traffic was planned to be applied in November and December 2002. Since the weather will be cooler during these two months no measurable additional rutting is anticipated.

Detailed reports are being prepared to document the details of the information provided in this report. The primary purpose of this report is to document the observations during the first cycle of the track. A more rigorous statistical analysis of the data and a more complete presentation of the data will be done in the final reports.

There were many variables that had to be considered for the analysis of the performance of the various test sections. Even with the high number of variables a number of trends in the data were observed that have provided information allowing one to improve the performance of HMA. Based on testing and observations during the first two years of traffic, a number of observations have been made. Some of these observations are identified below.

1. Use of moisture and temperature gauges was very successful. Over 80% of the gauges provided accurate results after 9 million ESALs.
2. Automatic belt sampling and mix sampling devices used during construction provided rapid, safe, representative samples.
3. Construction of short sections is very difficult. Good properties can be obtained if one pays attention to detail. The biggest problem in constructing a high quality HMA pavement surface at the Track was proper construction of the large number of transverse joints.
4. The trucking contract required that 10,000,000 ESALs be applied to the track resulting in approximately 1.6 million miles being driven. This provided a great opportunity to evaluate a number of items related to trucking.

5. Over a two-year period, the highest average seven-day maximum temperature was 61.4°C (142.6°F) at 20mm below the surface. This compares well with the expected temperature calculated using the Superpave procedures.
6. The amount of rutting in the test sections was negligible. The measurable rutting that was seen occurred in three stages. The first stage was the initial seating and compaction of the mix. The second and third stages were the two summers. Rutting essentially stopped when the seven-day average maximum air temperature was below 28°C. Rutting in the second summer (2002) was measurably less than that for the first summer (2001) even though the temperature was higher in 2002.
7. The highest surface temperature typically occurred at approximately 2:30 pm and the highest temperature at 10 inches below the surface typically occurred at approximately 10 pm showing a significant delay in heat transfer to the underlying layers.
8. Under traffic, the mixes using PG-67 asphalt binder densified more than the mixes using PG-76 asphalt binder. The binder layer for the mix with PG-67 densified more than the surface mix with PG-76. This may indicate that a little more binder can be used in the higher PG grade mixes to improve durability.
9. The amount of rutting calculated based on densification actually exceeded the actual measured rutting. This supports the fact that most of the test sections had very stable mixtures and the small amount of rutting that was measured was probably related to densification.
10. Diamond grinding was used on several transverse joints to improve the smoothness at the joint. This ground area was not sealed after grinding and actually performed very well. The grinding greatly improved the overall smoothness at the transverse joints.
11. The track roughness as quantified by the International Roughness Index increased slightly during two years of traffic. The IRI began in the mid 60's inches/mile and ended in the mid 70's inches/mile after two years.
12. Most mixes had an initial skid number above 50 and after two years ended in the 30s. One section that had an aggregate that polished, dropped below 20 and had to be overlaid.
13. The subgrade moisture quickly increased from about 10 percent during construction to about 25 percent after being covered. The moisture stayed relatively constant at about 25% in all of the sections for the two year period.
14. The amount of rutting was over 60 percent less in the sections with PG-76 than in sections with PG-67.
15. The performance of the coarse graded and fine graded mixes was approximately equal from a rutting point of view.
16. Adding an additional 0.5% asphalt binder increased the rutting in the PG-67 mixes by approximately 50% but had negligible effect on PG-76 mixes. Hence, it may be possible to design mixes, with higher PG grades, at slightly higher asphalt contents to improve durability.
17. The dynamic modulus test did not appear to be related to rutting. The confined repeated load test and the wheel tracking test did show some trend.
18. In general, all mixes performed well for two years. The mixes that had the higher rutting levels were mixes that had been designed to be susceptible to rutting. Even these mixes, that were designed to rut, had no significant rutting.

Traffic will continue on many of the sections for another two years so that additional information can be obtained to identify mixes that provide better performance and to determine laboratory tests that correctly quantify the performance.

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