EVALUATION OF LIMESTONE COARSE AGGREGATE IN ASPHALT CONCRETE WEARING COURSES

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by

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ABSTRACT

EVALUATION OF LIMESTONE AGGREGATE IN ASPHALT CONCRETE WEARING COURSES

Limestone is not used in asphalt concrete wearing courses in Alabama because of potential low skid resistance. Limestone is generally suspected to be polish susceptible and hence available sources of limestone are not utilized. This study was conducted to evaluate the performance of limestone when used in the coarse aggregate fraction of asphalt concrete wearing courses.

Three field test sites containing one experimental section each were constructed to study in-situ performance. The surface course mixes in experimental sections had 30 to 40 percent limestone in lieu of gravel in the coarse aggregate fraction. The control sections had 100% gravel as coarse aggregate for comparison. Performance of the surface courses was studied in the field by periodically measuring the skid resistance using the ASTM skid trailer and British Pendulum tester.

The in-situ mixes were replicated in the laboratory by using the same job mix formula and materials. Similar mixes were developed in the laboratory by varying the limestone content. Strength, moisture susceptibility and polishing properties mixes were measured in the laboratory. Acid insoluble tests, petrographic analysis and polish tests were also performed on the coarse aggregates to further understand their frictional and polish resisting properties.

Laboratory results show that the limestone in Fayette and Chilton County mixes are beneficial in increasing mix stability and resistance to moisture damage. However, the limestone in the Hale County mix did not

improve these properties. Laboratory frictional properties of limestone used in Chilton and Fayette Counties was comparable to the gravel aggregate. The limestone used in the Hale County was more polish susceptible. Field test results show that the experimental mixes with limestone had skid resistance comparable to the control gravel mixes in Fayette and Hale Counties. For Chilton County, skid numbers and British Pendulum numbers were higher on the experimental sections.

It appears possible to utilize some limestone in the coarse aggregate fraction without adversely affecting skid resistance. A maximum of 25% limestone coarse aggregate is recommended. Additional research to expand and verify the results from this study is needed.

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I. INTRODUCTION

Skid resistance of pavements has always been a major concern to pavement engineers. Skid resistance, along with structural capacity and rideability, is an essential feature of pavements. Since the aggregates make up more than 90% of the pavement structure, a majority of the pavement properties are significantly influenced by the choice of aggregate. When a pavement is opened to traffic, frictional resistance is initially offered by the pavement surface which is composed of asphalt coated aggregate matrix. With time asphalt films wear off and bare aggregates are exposed to traffic. Hence frictional properties of the surface are primarily controlled by characteristics of exposed aggregates. Utilization of aggregates exhibiting high frictional characteristics can therefore be viewed as a primary solution to the problem of skid resistance.

Many aggregates do not possess all the desirable properties. For example, siliceous gravels which generally show satisfactory skid resistance, are generally deficient in stability and stripping resistance. Carbonate aggregates usually possess good stability and stripping resistant but are often susceptible to polishing under traffic. Hence the use of carbonate aggregates from North Alabama deposits in surface mixes is restricted by the Alabama Highway Department (AHD).

Blending of aggregates results in a mix possessing qualities of all aggregates used in the blend. This has led pavement engineers to use aggregate blends which optimize the quality of the mix to make the pavement strong, durable and safe. Combinations of siliceous aggregate, with superior frictional properties, and carbonate aggregate, with superior structural properties, should provide structurally adequate pavements that allow safe vehicle operation.

The above stated factors served as motivation for this study. This research was conducted to investigate structural as well as polish and skid resistance characteristics of asphalt concrete wearing mixes when limestone aggregate is introduced in the coarse aggregate fraction (particles retained on #4 sieve size). Objectives and significance of the study are outlined in Chapter II. Chapter III describes the various factors affecting the pavement frictional properties related to the aggregates. A brief review on skid resistance measuring techniques is also included. Test sites, test plan, materials and tests performed are described in Chapter IV. Results and analysis of the data are presented in Chapter V. Conclusions and recommendations are presented in Chapter VI.

II. OBJECTIVE AND SIGNIFICANCE

OBJECTIVE

Large deposits of limestone are located in northern Alabama. Currently, these limestone sources are not used as coarse aggregates in asphalt concrete wearing courses because of suspected polish susceptibility. The AHD uses siliceous aggregates (primarily gravel) as coarse aggregate which sometime have to be hauled considerable distances. The objective of this research was to determine if limestone coarse aggregates could be used in asphalt concrete surface mixes without decreasing the long term skid resistance, and if durability and stability would be enhanced. Research efforts were directed towards comparing the skid resistance and structural properties of the mix when limestone was substituted for gravel in the coarse aggregate fraction of a mix. Proportions of limestone coarse aggregate were varied.

To meet the research objective, field and laboratory testing programs were set up in conjunction with the AHD. Three experimental projects utilizing limestone in the coarse aggregate fraction were constructed in Fayette, Chilton and Hale Counties. The performance of these projects were monitored and reported in this study. Based on the data from these projects and results obtained in the laboratory, recommendations are made relative to the use of limestone coarse aggregate in surface mixes.

SIGNIFICANCE OF THE STUDY

Skid resistance is of prime importance in design and construction of asphalt concrete pavements. Every year low skid resistance of roads causes loss of human life and property damage. Highway engineers try to minimize these losses by using highly skid resistant aggregates. Quite often artificial or synthetic aggregates, usually produced as industrial by-product (such as blast furnace slag), are used. Due to the depletion of locally available, naturally skid resistant aggregates and the high cost of synthetic aggregates, suitable aggregates may have to be imported to satisfy demand.

Blast furnace slag, which produces high stability and an excellent skid resistant surface was extensively used in the past in surface courses in Alabama. However, due to decreased production of blast furnace slag, siliceous gravel is now the predominate coarse aggregate in surface mixes. Blast furnace slag is highly absorptive, requiring high asphalt content, and has also contributed to decreased usage.

Siliceous gravel aggregates generally provide reasonable skid resistance when used in asphalt concrete pavements. However, stability and stripping are problems associated with their use. Due to their hydrophilic nature, siliceous aggregates have low affinity for asphalt and higher affinity for water. Stripping and ravelling of pavements occur due to this high affinity for water. Moreover, crushing of available size gravel often results in partially crushed particles which is thought to contribute to rutting.

Limestone aggregates are hydrophobic in nature, hence they have high affinity for asphalt and low affinity for water. This produces good bond with asphalt. High fatigue strength and stability are obtained due to this strong bond and the angular shape of the particles, resulting in stronger and more durable mix. However, limestones are generally suspected to be polish susceptible under the action of traffic leading to low skid resistance. Hence, abundant limestone deposits in North Alabama are not utilized in surface course mixes.

Some studies have shown that not all limestones exhibit poor skid resistant properties. Therefore limestone sources which show good skid resistant properties need to be identified. Also, some limestones having marginal skid resistance perform adequately when blended with aggregates of high skid resistance. Such blends also need to be evaluated and identified. Results confirming these suppositions will allow discriminatory use of limestones in surface mixes to improve stability and durability without significantly affecting the skid resistance.

III. BACKGROUND AND LITERATURE REVIEW

Various factors affect the skid resistance of asphalt concrete pavements. The prominent factors are :

1) Traffic,
2) Vehicle speed,
3) Weather conditions, and
4) Pavement surface condition.

Though all the above are equally important, this study focussed on factors affecting pavement surface conditions; the primary element being the aggregate used in the asphalt concrete mix. The factors affecting the frictional properties of the pavement surface and a brief discussion on measurements of skid resistance is presented in the following sections.

MICROTEXTURE

Frictional properties of aggregates are basically derived from the texture of their surface. The texture or roughness of individual particles less than 0.5 micron is termed microtexture (1). This characteristic of individual particles is also called harshness of the surface. The microtexture allows for penetration of water films and provides contact area for the tire. This increases the tire grip on the pavement surface. The microtextural features of an aggregate govern the skid resistance properties of the pavement at low speeds.

The microtexture of the pavement surface can be measured with the British pendulum tester or some other profiling techniques (1). The British pendulum tester is widely used for determining microtexture of pavement surface or aggregates. Microtexture is often attributed to porosity or vesicular properties of the individual aggregate. Artificial aggregates like slag have excellent microtexture hence they possess good frictional properties, and yield skid resistant pavements when used in surface mixes.

MACROTEXTURE

Surface irregularities or surface topography greater than 0.5 micron is termed as macrotexture. The macrotexture of a surface is formed by the aggregate matrix of the mix. It is expressed as texture depth in inches or mm (1). The macrotexture of a surface is influenced by aggregate orientation, size, and gradation. Macrotexture of a surface is essential since it provides drainage channels for water at the tire pavement interface. It reduces hydroplaning and therefore maintains skid resistance on wet pavements. This is very important at medium and high speeds (more than 40 mph). Skid trailers or stereophotography are usually employed to measure the macrotexture of a pavement surface. Outflowmeter and sand-patch methods are some other methods to measure surface macrotexture (2).

The surface should have sufficient macrotexture to maintain adequate skid resistance. However, excessive macrotexture can result in low surface contact area for the tire, and can be detrimental to the skid resistance of the pavement. Large macrotexture can also increase tire wear and noise levels.

SIZE

As aggregate size increases, the macrotexture of the pavement surface increases. The void depth or peaks per unit length can be increased by proper selection of size of the aggregate. This results in better drainage channels at the tire-surface interface. Variation in maximum size can result in changes in the pavement friction under equivalent traffic (2).

Aggregates are subjected to imbedment under the action of traffic and may cause bleeding. Bleeding makes the pavement surface slippery. If large size aggregate are chosen, the aggregates on the whole have better imbedment resisting capacity. This improves maintenance of macrotexture over a longer period (3).

SHAPE

The shape of aggregate is also an important consideration in designing a skid resistant pavement. Most rounded aggregates are natural aggregates transported and deposited by water. These aggregates do not provide good macrotexture.

Angular aggregates provide good skid resistance as long as they maintain their angularity. Under the action of traffic, sharp edges are often lost and this results in reduced roughness and macrotexture. If the loss of angularity or sharpness is differential or renewable then it is a beneficial phenomenon. Differential wearing is a function of mineralogical composition of the aggregates.

GRADATION

It is clear that aggregates play a very important role in providing skid resistance to the pavement. The amount or size of the aggregate components can be controlled by varying the gradation of the mix. Gradation of the mix affects the surface texture of the pavement to a considerable extent. The option of controlling the macrotexture by varying the gradation of the mix has led to the construction of open-graded friction courses (1). Textures of surface courses have been successfully altered by changing mix gradation (4). Higher coarse aggregate content in mixes can also lead to longer lasting macrotexture (3). Increased void depth and decreased imbedment of the aggregate is usually associated with higher coarse aggregate content. Though increased coarse aggregate content leads to better skid resistance and structural strength, rideability criteria should not be compromised.

Fines or fine aggregates also play an important role in skid resistance (5). Hard siliceous sand was found to produce good skid resistant pavement surface, while dolomitic limestone fines produced poor pavement skid resistance (6). Under the action of traffic most of the fines are worn away leaving coarse aggregate exposed. Hence coarse aggregate characteristics tend to be more important in long-term skid resistance.

MINERALOGY

Mineral composition is another factor which affects the frictional properties of aggregates. The mineral composition of the majority of aggregates are heterogenous in nature. Individual aggregates are composed of a variety of minerals and fossils. The frictional and polishing properties of these aggregates are affected by the hardness of minerals present (7,8). The presence of hard minerals (Mohs hardness > 5) is vital for polish resistance. Soft minerals in the aggregate polish at a fast rate under traffic, while the hard mineral have better resistance to polish. Though the hard minerals resist polishing in the initial stages, they eventually become smooth, losing their frictional properties (9).

Combinations of hard and soft minerals leads to differential wear and polishing thereby maintaining a rough surface. It has been observed that aggregates having 40 to 70 percent hard minerals show higher polish resistance than those having hard mineral composition outside this range (10).

Petrographic analysis of aggregates can be done by visual observation of individual aggregates and by thin section methods. In a typical petrographic analysis, mineral composition, hardness and softness of individual minerals, fossils, cleavage, porosity, etc. are determined. It has also been observed that the aggregates which possess high porosity have harsh surface. These aggregates usually have sharp edges which increases roughness. Hence, from the mineralogy or petrographic properties of the aggregates, it may be possible to estimate frictional properties.

BLENDING

Blending is a practical and economical way of providing adequate skid resistance. Blending of superior aggregates with those possessing poor frictional properties can give overall acceptable skid resistance. A blend of limestone which generally has marginal skid resistance and silica gravel which generally possesses good skid resistance has provided skid resistance (4). The polish resistance of the blend was found to be affected in the same proportion as the percentages of the two aggregates in the blend. Blending is used when superior aggregates are limited in supply and/or costs are high. Synthetic aggregates like slag which possess excellent skid resistance properties are often blended with natural aggregates. This facilitates saving on the synthetic aggregates and utilization of the aggregates possessing poor skid resistance. Blends of aggregates possessing high and low polish resisting properties result in differential wearing of the surface which is beneficial in maintaining the skid resistance of the pavement surface (11). Blending of aggregates with known polish resisting characteristics result in improved skid resistance of pavements and results are often predictable and controllable (12).

POLISHING AND WEARING

Polishing of the pavement surface under the action of traffic result in slippery pavements. Polishing decreases the microtexture of the surface. Wearing involves loss of material on the surface and results in increased roughness. Polishing on the other hand does not involve loss of material (4).

Wearing is beneficial to the surface as long as it occurs in controlled amount. If excessive wearing takes place, the pavement loses its rideability quality and tires are damaged. Differential wearing to a certain extent is most desirable since it maintains harsh surfaces. Wearing of the surface depends on the mineralogy of the aggregates, volume of the traffic, type of tire, surface condition and presence of dust/or grit on the pavement surface.

Polishing reduces aggregate surface irregularities. Soft surfaces polish at a faster rate than harder surfaces. Hard surfaces which initially resists polishing finally become smooth due to reduction in microtexture. Polishing is strictly an aggregate surface phenomenon and largely depends on aggregate characteristics. The volume of traffic, aggregate properties and dust and/or grit on the surface are some of the factors affecting the rate of polishing.

In the laboratory, polishing and wearing under the action of traffic is simulated by using the circular track or jar mill method (13). Rotary drum polish machines or modified drum machines are also used to polish aggregate specimens (14). Different states and agencies use different kinds of polish machines to simulate traffic actions (12,13). Loss in friction due to reduction in microtexture is usually measured by the British portable tester.

The British polish wheel is the most widely used polish machine. The British polish machine shown in Figure 1 consists of a 18"-diameter wheel. Aggregate specimen 3.5" long, 1.7" wide and 0.63" deep (Figure 2) are prepared in accordance with ASTM D 3319-83 and are fitted on the rim of the polish wheel. A pneumatic tire rests on the wheel. The wheel rotates at about 320 rpm. The pneumatic tire resting on the wheel rim then polishes the specimens fitted on the rim. Water and grit (No. 150 silicon carbide) is fed through a chute to accelerate polishing and simulate the action of abrasives on the pavement surface. Water also serves as a coolant to reduce the heat generated during the polishing.

Attempts to polish the asphalt mix specimens in the laboratory were made in this study. A literature review showed that not many researchers have attempted to polish asphalt mix samples on the British polish wheel.

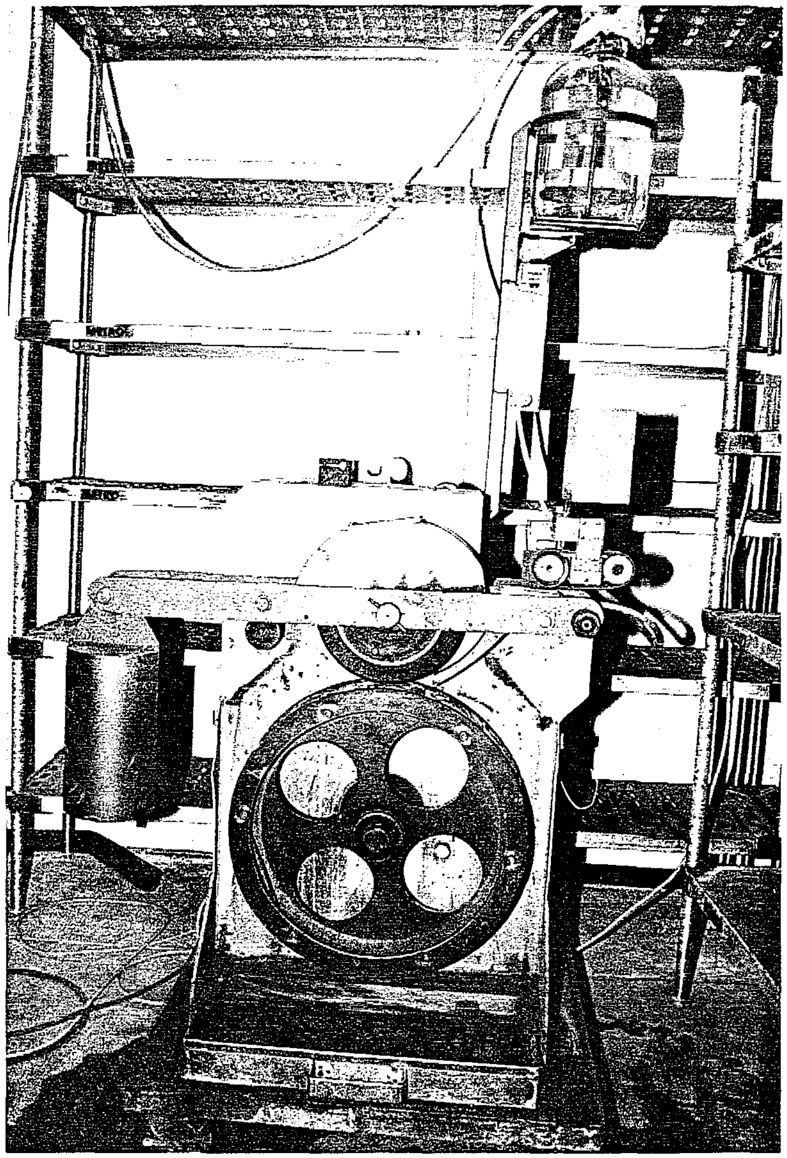

Figure 1. British polish machine

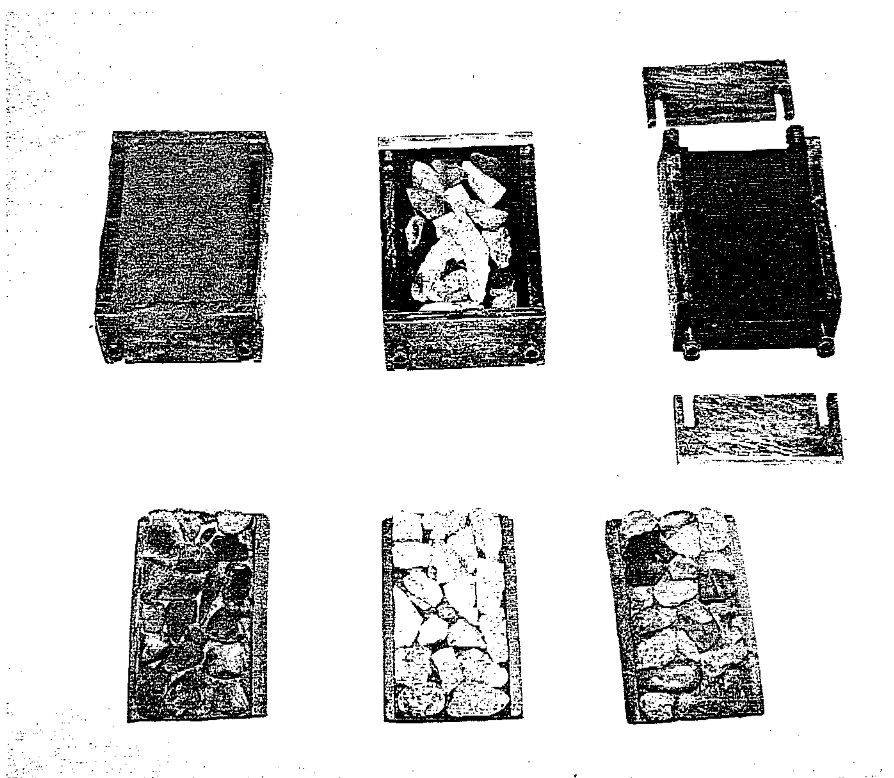

Figure 2. Polish specimens and molds

The procedure for preparing asphalt mix specimens for the British polish wheel was adopted from a study conducted in Texas (15) and modified as discussed later.

FIELD MEASUREMENTS

There are various methods of measuring pavement skid resistance. The primary objective is to measure the resistance offered by the pavement surface (when wet) to a tire which is prevented from rolling.

Skid resistance of pavements is most often determined using the locked-wheel skid trailer (ASTM E249). The California skid tester and the Pennsylvania State University's drag tester are two of a variety of testers used in the field to directly measure skid resistance of pavements (4,18).

The locked-wheel skid trailer (Figure 3) consists of a truck pulling the skid trailer. The skid trailer is usually two-wheeled. Both the tires conform to the required specification (ASTM E 249). The trailer is generally towed at 40 mph. At times, the trailer is towed at a speed higher or lower than the standard 40 mph to determine the speed gradients (19). Lower skid resistance is recorded at higher speeds. Water is jetted on the surface in front of the test tire when the tire reaches the test speed. This is done in order to simulate the wet conditions (rain). Either one or both wheels are locked to measure the skid resistance of the pavement surface. When the test wheel is locked, the trailer is dragged by the truck. The resistance offered by the pavement surface is measured by a torque measuring device in the trailer. This resistance is converted into a numerical value called skid number (SN).

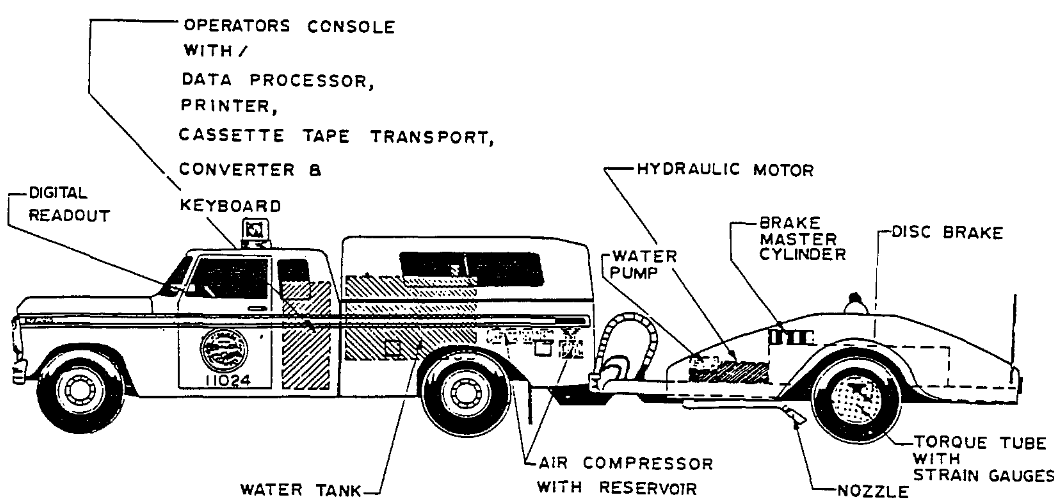

Figure 3. Typical skid trailer

The texture of the surface is an indirect method and will give some idea of the resistance offered by the surface to a tire which is prevented from rolling. Surface texture can be measured by the sand patch method, outflow meter and stereophotography (25). Surface roughness measured by profilometer and Mu-meter also provide an indication of the skid resistance of pavement surfaces (16,17).

LABORATORY MEASUREMENTS.

In the laboratory, relative skid and polish resistance can be estimated on the basis of frictional properties of aggregates and asphalt mixes. The frictional characteristics of bare aggregates are measured using the British portable tester (4,18).

The British Pendulum Tester was developed by the British Road Research Laboratory and is used widely throughout the United States and Europe. It operates on the principle of energy conservation. The tester has a pendulum arm which has a rubber slider attached. The rubber slider, of size 0.25" by 1" by 1.25", is used to test a curved polish wheel specimens. The composition of the rubber slider is specified in ASTM E303. The pendulum arm is allowed to slide over the surface tested. The pendulum swing is inversely proportional to the roughness of the surface tested. The swing of the pendulum is recorded by the indicator arm swinging along with the pendulum arm and recorded as a British pendulum number (BPN). If the surface is rough, the indicator points to a higher BPN value on the scale and vice versa.

The British pendulum tester can be used in the field as well as in the laboratory. A 1/4" by 1" by 3" size slider is used when measurements are taken in the field. Cores or slabs cut from the pavement can also be tested in the laboratory using this instrument. The pendulum swings at approximately 5 to 7 mph and hence the correlation with skid trailer which measures skid resistance at 40 mph can be expected to be quite poor (4).

CURRENT PRACTICES

In order to obtain current practices and strategies for providing resistant pavements a questionnaire was sent to various state highway officials. The state highway agencies were asked to indicate current criteria used to pre-evaluate coarse and fine limestone aggregates for asphalt concrete surface mixes. Responses obtained from the various states are tabulated in Table 1. This survey indicated that most of the States have not formulated definitive skid resistance guidelines, but there is constant monitoring and collection of skid data.

A literature review shows that the BPN recorded on various bare aggregates range from 21 to 75 (19). This wide range could be attributed to the various polish methods implemented to simulate the polishing action of the traffic. France requires a BPN of 45 or higher to rate aggregate as good. The British specifications require the aggregate have a BPN range of 45 to 75 to be accepted in the wearing course mix. Higher values are usually specified for heavily trafficked roads. Great Britain and France also specify texture depth as one of the acceptance criteria (17). Japan requires braking force coefficient yield maximum of 0.40 for normal traffic and 0.45 for critical curves and intersections.

Table 1

Summary of skid resistance criteria used in United States.

State	Requirements
Delaware	Minimum SN of 35.
Florida	Polish susceptible limestone not used. Percent insoluble residue and SN determined where limestone is used.
Illinois	Periodic monitoring.
Iowa	Petrographic examination is conducted on aggregates. Periodic SN are recorded.
Kansas	SN recorded periodically but no specification implemented.
Kentucky	10% minimum insoluble residue in limestone aggregates. Gravel should have minimum of 20% soluble residue. Depending on volume of traffic and speed, percentage of skid resistant aggregate required (20 to 50%).
Louisiana	Uses BPN and SN to monitor skid resistance of roads.
Maryland	Minimum BPN of 23 is recommended.
Massachusetts	Skid test conducted but not implemented, a minimum SN of 40 is desired.
Michigan	Aggregate wear index based on petrographic analysis of crushed material used in the mix.
Minnesota	Limestone is not used on high AADT roads. Percent insoluble residue is determined in some districts where limestone is used.
	.......contd

Table 1 ...contd

Mississippi	SN of 35 or more is desirable.
Montana	L.A.Abrasion test and petrographic analysis are used to pre-evaluate aggregates.
New York	Minimum of 20% noncarbonate material, petrographic analysis and minimum SN of 32 after 10 million passes.
Ohio	SN of 35 used as a informal guide.
Oregon	Specification under evaluation. An informal SN target of 37.
Pennsylvania	Aggregates rated on basis of ASTM C295, petrographic analysis, accelerated polish test, ASTM D3042 or PTM No.618. SN of 30-35 given an informal OK. In-depth study of pavements conducted if the SN falls below 30.
Puerto Rico	At least 40% on L.A. Abrasion test. A minimum BPN of 48 is specified.
Rhode Island	Limestone used in mixes should meet the L.A. Abrasion and sodium sulphate soundness test requirements.
Texas	Uses BPN/Polish Stone Value (PSV) to evaluate the aggregates in mixes.
Vermont	Percent acid insoluble residue and percent wear based on L.A. Abrasion test is determined.

.......contd

Table 1 . . . . contd

West Virginia	Aggregates tested for BPN/PSV, petrographic examination and percent acid insoluble residue.
Wisconsin	SN of 30 or more. SN of 35 for high traffic volume.

IV STUDY DESCRIPTION

Three experimental field projects constructed by the Alabama Highway Department were used in this study. Each project namely, SR 13 in Fayette County, US 31 in Chilton County and SR 25 in Hale County included a one mile experimental section which had a specified percentage of limestone (in lieu of gravel) in the coarse aggregate fraction (particles retained on the #4 sieve) of the wearing course. The remaining pavement of the projects contained 100% gravel coarse aggregate for comparing frictional characteristics. The surface course/mixes using 100% gravel as coarse aggregate will hereinafter be referred as control section/mixes, and those using limestone as part substitution for gravel coarse aggregate fraction will be referred as experimental section/mixes.

Following are locations of each project :

	Mile Post	
County	Entire Project	Experimental section
Fayette	233.2 to 250.0	248.0 to 249.0
Chilton	235.0 to 241.0	236.0 to 237.0
Hale	37.1 to 51.0	48.0 to 49.0

Following are the percentages of limestone in the coarse aggregate fraction of the experimental mixes.

County	Composition of coarse aggregate fraction
Fayette	40% Limestone + 60% Gravel
Chilton	30% Limestone + 70% Gravel
Hale	33% Limestone + 67% Gravel

Traffic data for all three counties is tabulated in Table 2. The traffic data as obtained in August 1989. The traffic breakdown on commercial, heavy and medium vehicle for Hale County was not available.

The 30 to 40% limestone in the coarse aggregate fractions do not provide data with a range sufficient to develop a criteria for including limestone in surface mixes. Therefore, additional mixes were developed in the laboratory to include a wider range of percent limestone in the coarse aggregate fraction. Two additional limestone contents of 20 and 60% were selected. The job mix formula used for each blend conformed to Alabama Highway Department surface course specification.

Aggregate, asphalt and asphalt mix samples were collected for each project during construction. These samples were used in the laboratory evaluation of the mixes. Available data on job mix formula, aggregate and asphalt properties were also obtained.

The AHD skid trailer and British pendulum tester were employed to measure in-situ skid resistance. Skid trailer and British pendulum tester measurements were taken at the same spot to attempt correlations between the two instruments. Surface and ambient temperatures were also recorded.

Skid numbers generally vary with seasons. Skid numbers are likely to be high during late fall and winter, and low during late spring and summer (19). Field measurements were taken during summer and winter providing a wide range of pavement temperatures.

The Marshall mix design procedure was used to determine the optimum asphalt content of the various mixes. Marshall stability and indirect tensile strength test were conducted on the mixes. Marshall

Table 2.

Traffic data for all three counties for 14th August 1989

SR 13 FAYETTE COUNTY

Mile Post	AADT	LT	ST	%CV	%HV	%MD
233.4 - 237.7	970	1.025	1.025	26	70	30
237.7 - 242.0	1020	1.025	1.025	26	70	30
242.0 - 245.2	1000	1.025	1.025	27	70	30
245.2 - 247.7	1330	1.025	1.025	29	70	30
247.7 - 250.9	1190	1.025	1.025	32	70	30

US 31 CHILTON COUNTY

Mile Post	AADT	LT	ST	%CV	%HV	%MD
233.4 - 235.5	4100	1.020	1.030	10	70	30
235.5 - 240.0	3210	1.020	1.030	10	70	30
240.0 - 241.3	4080	1.020	1.030	7	70	30

SR 25 HALE COUNTY

Mile Post	AADT	LT	ST	%CV	%HV	%MD
37.0 - 40.0	780	1.010	1.010	NA	NA	NA
40.0 - 44.0	790	1.010	1.010	NA	NA	NA
44.0 - 47.0	860	1.010	1.010	NA	NA	NA
47.0 - 50.0	810	1.010	1.010	NA	NA	NA
50.0 - 51.0	1680	1.010	1.010	NA	NA	NA

AADT = Annual Average Daily Traffic
LT = Long Term Growth
ST = Short Term Growth
CV = Commercial Vehicles
HV = Heavy Vehicles
MD = Medium Vehicles
NA = Not Available

stability was determined to evaluate the strength and deformation resistance of the mix. Effects of moisture conditioning were determined by using a "modified" Lottman test. The modification referring to the application of only one freeze-thaw cycle compared to eighteen cycles in the original Lottman procedure.

Polish resistance of bare aggregates and asphalt mixes were evaluated using the British polish wheel (ASTM D3319) and British portable tester. The asphalt mixes were tested for polish resistance as they were more representatives of the pavement surface than the bare coarse aggregates. This may also provide an alternative which will allow a complete evaluation of a mix including fine and coarse aggregate properties and proportions as well as asphalt content.

Insoluble Residue Test for Carbonate Aggregates (ASTM D 3042-86) and petrographic analysis of aggregates were included to provide additional information on the composition of the limestone aggregates that might support field and laboratory test results.

Modified Lottman test was used to determine the moisture resistance of the mixes. Marshall samples were prepared for each mix at the selected optimum asphalt content. These samples were compacted at various Marshall hammer blows to determine the number required to obtain about 7% air voids. Six samples were prepared for each mix at approximately 7% air voids. These samples were divided into two sets of three such that the average air voids of the sets were nearly equal. One set was kept dry for control. The other set (wet) was subjected to vacuum while submerged to obtain 60-80 % water saturation. The samples of the wet set were then frozen for 16 hours. After freezing, the samples were placed in water at 140^0 F for 24 hours. Both sets of specimens were then placed in bath at 77^0 F for 2 hours, and tested for indirect tensile strength. The TSR which is the ratio of average tensile strength of the wet set to the average tensile strength of the dry set was then determined.

FAYETTE COUNTY

This project was constructed in April 1988. Forty percent limestone was used in the coarse aggregate fraction of the experimental mix. Details of the materials, mixes and job mix formulas for control and experimental sections used in the field are shown in Tables 3 and 4, respectively. Mixes with 20 and 60% limestone coarse aggregate were also prepared for laboratory testing by varying the amounts of #6 and #7 limestone and #78 crushed gravel.

Optimum asphalt contents (4% air voids) of 5.5% and 5.15% were obtained by the AHD for the control and experimental mix, respectively. These asphalt contents were obtained using Marshall mix design procedure with 50 blows of a static base mechanical hammer. In the laboratory, optimum asphalt contents of 5.5, 5.2, 5.1 and 5.0% were obtained, respectively, for the control, experimental, 20% and 60% limestone coarse aggregate mixes.

CHILTON COUNTY

This project was constructed in November 1987. The experimental section contained 30% limestone in the coarse aggregate fraction of the wearing course mix. Details of aggregates and asphalt mixes for the

Table 3.

Job mix formula for control section of Fayette County

County : Fayette (SR 13)
State mix type : Section 416, Mix "B"

Aggregates:	15%	#6 Crushed Gravel	Blue Ridge S&G Co., Winfield,AL
	30%	#78 Crushed Gravel	Blue Ridge S&G Co. Winfield, AL
	47%	Coarse Sand	Blue Ridge S&G Co. Winfield, Al
	8%	Agg Lime	Dolcito Quarry, Tarrant, AL
Asphalt Cement:		AC-20	Hunt Oil Co., Tuscaloosa.

Job Mix Formula :

SIEVES	% PASS
2"	
1	100
3/4	97
1/2	87
3/8	80
4	61
8	45
30	30
50	17
100	6
200	4.4

Other information :

1.	% asphalt cement required	:	5.5
2.	Maximum specific gravity	:	2.321
3.	% Anti strip	:	0.5-1.00
4.	Stability	:	1670
5.	% absorption of coarse aggregate	:	2.3
6.	Apparent specific gravity of agg.	:	2.608
7.	Effective specific gravity of agg.	:	2.503
8.	Bulk specific gravity of agg.	:	2.493
9.	Specific gravity of AC	:	1.033

Table 4.

Job mix formula for experimental section of Fayette County

County : Fayette (SR 13)
State mix type : Section 416, Mix "B"

Aggregates:	15%	#6 Limestone	Vulcan Matls., Russellville, AL
	5%	#7 Limestone	Vulcan Matls., Russellville, AL
	30%	#78 Crushed Gravel	Blue Ridge S&G Co. Winfield, AL
	40%	Coarse Sand	Blue Ridge S&G Co. Winfield, Al
	10%	Agg Lime	Dolcito Quarry, Tarrant, AL
Asphalt Cement:		AC-20	Hunt Oil Co., Tuscaloosa.

Job Mix Formula :

SIEVES	% PASS
2"	
1	100
3/4	99
1/2	87
3/8	82
4	60
8	42
30	29
50	20
100	8
200	5.2

Other information:

1. % asphalt cement required : 5.15
2. Maximum specific gravity : 2.358
3. % Anti strip : 0.5-1.00
4. Stability : 1910
5. % absorption of coarse aggregate : 2.4
6. Apparent specific gravity of agg. : 2.643
7. Effective specific gravity of agg. : 2.534
8. Bulk specific gravity of agg. : 2.506
9. Specific gravity of AC : 1.033

control and experimental section wearing courses are tabulated in Tables 5 and 6, respectively. Mixes with 20 and 60% limestone coarse aggregate were also prepared for laboratory testing by varying the amount of #6 and #78 limestone and 1/2" crushed gravel.

The 50 blow Marshall mix design procedure was used to determine the optimum asphalt content for control, experimental, 20% and 60% limestone mixes. The optimum asphalt content of 5.2% was obtained for all mixes.

HALE COUNTY

This project was constructed in July 1990. The experimental section contained 33% limestone in the coarse aggregate fraction of the wearing course mixes. Details of the control and experimental wearing course mixes are tabulated in Tables 7 and 8, respectively. Mixes with 20 and 60% limestone coarse aggregate were also prepared for laboratory testing by varying the amount of #78 limestone and 1/2" crushed gravel.

The Marshall mix design procedure was conducted and optimum asphalt contents of 5.9, 5.5, 6.1 and 4.8% determined for the control, experimental, 20% and 60% limestone mixes, respectively.

IN-SITU SKID MEASUREMENTS

The AHD skid trailer was used to measure the skid number (SN) for all three counties during winter and summer to assess temperature effects (19). All pavements were two lane and Sns were measured on both lanes. Readings were taken in each mile on the control section and in every 2/10th of mile on the experimental section. The center of locations where the skid trailer was locked were marked, and readings of the British pendulum tester were taken at these locations.

Table 5.

Job mix formula for control section of Chilton County

County : Chilton (US 31)
State mix type : Section 416, Mix "B"

Aggregates:	45%	3/4" down washed Crushed Gravel	Wade and Vance, Jemison, AL
	13%	1/4"-0" Screenings	Dravo Basic Mat'ls Saginaw, AL
	35%	Coarse Sand	S & S Materials Selma, AL
	7%	Sump Lime	Southern Ready Mix, Calera, AL
Asphalt cement :		AC-30	Hunt Oil Co., Tuscaloosa, AL

Job Mix Formula :

SIEVES	% PASS
2"	
1	100
3/4	100
1/2	90
3/8	75
4	55
8	46
30	29
50	11
100	8
200	5.7

Other information :

1.	% asphalt cement required	: 5.2
2.	Maximum specific gravity	: 2.457
3.	% Anti strip	: 0.5-1.00
4.	Stability	: 1930
5.	% absorption of coarse aggregate	: 1.0
6.	Apparent specific gravity of Agg.	: 2.663
7.	Effective specific gravity of Agg.	: 2.657
8.	Bulk specific gravity of Agg.	: 2.612
9.	Specific gravity of AC	: 1.036

Table 6.

Job mix formula for experimental section of Chilton County

County : Chilton (US 31)
State mix type : Section 416, Mix "B"

Aggregates:	15%	#6 Limestone	Southern Ready Mix Calera, AL
	5%	#78 Limestone	Southern Ready Mix Calera, AL
	45%	1/2" down crushed gravel	Wade & Vance and Central AL Pav., Calera, AL
	22%	Coarse Sand	Superior Products Inc., Jemison, AL
	13%	Agg Lime	Southern Ready Mix Calera, AL
Asphalt cement :		AC-20	Hunt Oil Co., Tuscaloosa, AL

Job Mix Formula :

SIEVES	% PASS
2"	
1	100
3/4	99
1/2	89
3/8	77
4	55
8	45
30	30
50	14
100	8
200	5.8

Other information :

1.	% asphalt cement required	5.15
2.	Maximum specific gravity	2.475
3.	% Anti strip	0.5-1.00
4.	Stability	2035
5.	% absorption of coarse aggregate	1.4
6.	Apparent specific gravity of Agg.	2.693
7.	Effective specific gravity of Agg.	2.678
8.	Bulk specific gravity of Agg.	2.629
9.	Specific gravity of AC	1.033

Table 7.

Job mix formula for control section of Hale County

County : Hale (SR 25)
State mix type : Section 416, Mix "1"

Aggregates:	60% 1/2" down Crushed Gravel	Asphalt Contractors Selma, AL
	15% M-10 Limestone	Dravo Basic Matls Maylene, AL
	25% Blended Natural Sand	Asphalt Contractors Inc., Selma, AL
Asphalt cement :	AC-20	Hunt Oil Co., Tuscaloosa, AL

Job Mix Formula :

SIEVES	% PASS
2"	
1	
3/4	100
1/2	97
3/8	88
4	67
8	53
16	44
30	35
50	22
100	10
200	6.5

Other information :

1.	% asphalt cement required	5.90
2.	Maximum specific gravity	2.433
3.	% Anti strip	NA
4.	Stability	2091
5.	% absorption of coarse aggregate	NA
6.	Apparent specific gravity of Agg.	2.662
7.	Effective specific gravity of Agg.	2.658
8.	Bulk specific gravity of Agg.	2.626
9.	Specific gravity of AC	1.033

Table 8.

Job mix formula for experimental section of Hale County

County : Hale (SR 25)
State mix type : Section 416, Mix "1"

Aggregates:	20%	#78 Limestone	Dravo Basic Matls. Maylene, AL
	40%	1/2"-Down Coarse gravel	Asphalt Contractors Inc., Selma, AL
	25%	Concrete Sand	Asphalt Contractors Inc., Selma, AL
	15%	Agg Lime	Dravo Basic Matls, Maylene, AL
Asphalt cement:		AC-20	Hunt Oil Co., Tuscaloosa, AL

Job Mix Formula :

SIEVES	% PASS
2"	
1	
3/4	100
1/2	97
3/8	86
4	61
8	51
16	44
30	33
50	15
100	8
200	5.5

Other information :

1.	% asphalt cement required	5.45
2.	Maximum specific gravity	2.492
3.	% Anti strip	NA
4.	Stability	2253
5.	% absorption of coarse aggregate	NA
6.	Apparent specific gravity of Agg.	2.720
7.	Effective specific gravity of Agg.	2.712
8.	Bulk specific gravity of Agg.	2.663
9.	Specific gravity of AC	1.033

Three sets of BPN readings were taken at each location. These readings were taken at the location of the skid trailer test mark and on either side of the marking, at approximately 15 feet. This encompassed the length of pavement over which the skid trailer wheel remained locked and gave an overall average of the pavement texture. This facilitated correlations between the SN and the BPN. Surface and ambient temperatures were periodically monitored during testing.

LABORATORY TESTING OF AGGREGATES FOR SKID RESISTANCE

Bare aggregates were polished in the laboratory on the British polish wheel and then tested with the British pendulum tester. The British polish wheel used silicon carbide grit (150-grit size) to accelerate polishing and simulate actions of the abrasives on the pavement surface. Water was fed at the rate of 50 to 60 ml per minute to reduce the heat produced during the polishing. Bare aggregate samples passing 1/2 inch and retained on the 3/8 inch sieve size were used. The original BPNs of the polish specimens were recorded before fitting them on the British polish wheel. The samples were polished for a total of 9 hours. The samples were dismounted at 3-hour intervals and BPNs recorded to obtain the loss in frictional properties with polishing time.

Blends of gravel and limestone were polished on the polish wheel and tested by the pendulum tester to replicate coarse aggregate blends used in the mixes. Pieces of gravel and limestone were placed randomly in the specimen mold. The exposed aggregate surface areas were traced on tracing paper with a fiber tip pen, and limestone and gravel aggregates identified. The percentage of limestone area on the specimen surface was determined using a planimeter.

LABORATORY TESTING OF ASPHALT MIX SPECIMENS FOR SKID RESISTANCE

Marshall samples of mixes were prepared at optimum asphalt content. Marshall samples were subjected to 75 blows per face in order to obtain a slice which could withstand the polishing and impacts under the British polish wheel rubber tire. No hand tamping with spatula was done prior to applying 75 blows. It was observed that the untamped samples provided relatively more coarse aggregates on the faces than those samples which were tamped. The face of the sample which was compacted first was marked as top face and the other was marked as bottom face. This distinction was made for two reasons. Firstly, such a procedure was adopted to find out the face which provided larger amount of coarse aggregate exposed on its face and secondly, there was a possibility that the top and bottom surfaces might exhibit different friction levels.

The Marshall samples were sliced with a diamond saw to provide two 1/4 inch thick circular slabs from top and bottom; both faces having one side sawed. The middle section of the Marshall samples which had both its sides sawed were also tested. The sawing of the sample subjected the side being sawed to a considerable degree of polishing. It was speculated that the friction level of the sawed surfaces might be close to the final or terminal friction level that the mix could attain. The circular slabs of the mix were then cut into rectangular specimens to fit into the polish specimen mold as shown in Figure 2.

Each rectangular slab was identified by its county, type of mix, sample section (top, bottom or middle) and sample number. The rectangular slabs along with the mold were placed in an oven at 100^0 F for 1-2 hours so that the samples deformed slightly and conformed to the curvature of the mold. The specimens were then cooled to room temperature. Epoxy was applied to form the base for the specimens and cured for at least 6 hours. The asphalt mix polish specimens were then mounted on the British polish wheel and tested in the same manner as the bare aggregate polish specimens.

ACID INSOLUBLE TEST FOR LIMESTONE AGGREGATES

This test was conducted in order to determine the amount and size of noncarbonate particles in the limestone aggregates. This test is used by some states to control the carbonate content in surface mixes. New York discourages the use of carbonate aggregates with less than 10% sand-sized impurities (material insoluble in acid). Mixtures of carbonate and noncarbonate aggregates are required to have at least 20% insoluble material in the mixture (21). Pennsylvania gives excellent rating to aggregates with carbonate content less than 10% (15).

Sand-sized impurities play a very important role in developing the skid resistance of an aggregate surface. The coefficient of friction of surfaces increases with increasing percent of insoluble residue in the larger sized aggregates (6). Although this test is a good guideline to evaluate the polish resistance of aggregates, AASHTO recommends consideration of field experience along with test results (17).

The acid insoluble test was performed in accordance with procedure outlined in ASTM D 3042-86. Only the limestone aggregates of all three counties were evaluated. The gravel aggregates were assumed to have very small if any carbonate content. Three samples weighing 500 grams each were tested for each limestone type. Six normality HCL solution was used to dissolve the carbonate material as outlined in the test procedure. The insoluble residue was washed over a #200 sieve and the percentage retained determined.

V. PRESENTATION AND ANALYSIS OF RESULTS

MARSHALL MIX DESIGN TEST RESULTS

A discussion of the Marshall mix design data for all three counties is presented in this section. The results of the Marshall stability tests are summarized in Figure 4.

Fayette County: The Marshall mix design data for Fayette County are summarized in Table 9. The air voids at design asphalt contents (5.5 and 5.2%) were higher than 4% used by the AHD during the mix design for the control and experimental mixes, respectively. The decision was made to keep the asphalt contents the same as those used by the AHD for control and experimental mixes. The asphalt content for the 20% and 60% limestone sets were chosen to give the same air voids as the experimental mix (40% limestone). The following are the optimum asphalt contents for all mixes:

Control set	:	5.5%
20% limestone set	:	5.1%
40% limestone set (Experimental)	:	5.2%
60% limestone set	:	5.0%

The asphalt contents shown above gave air voids of about 5.5% for all four mixes. Marshall stability values were estimated for each mix at the above asphalt contents and plotted in Figure 4. The concave upward relationship indicates that there is a slight decrease in stability for 20% limestone coarse aggregate, that the stability for 40% limestone is about the same as the control mix and that the stability

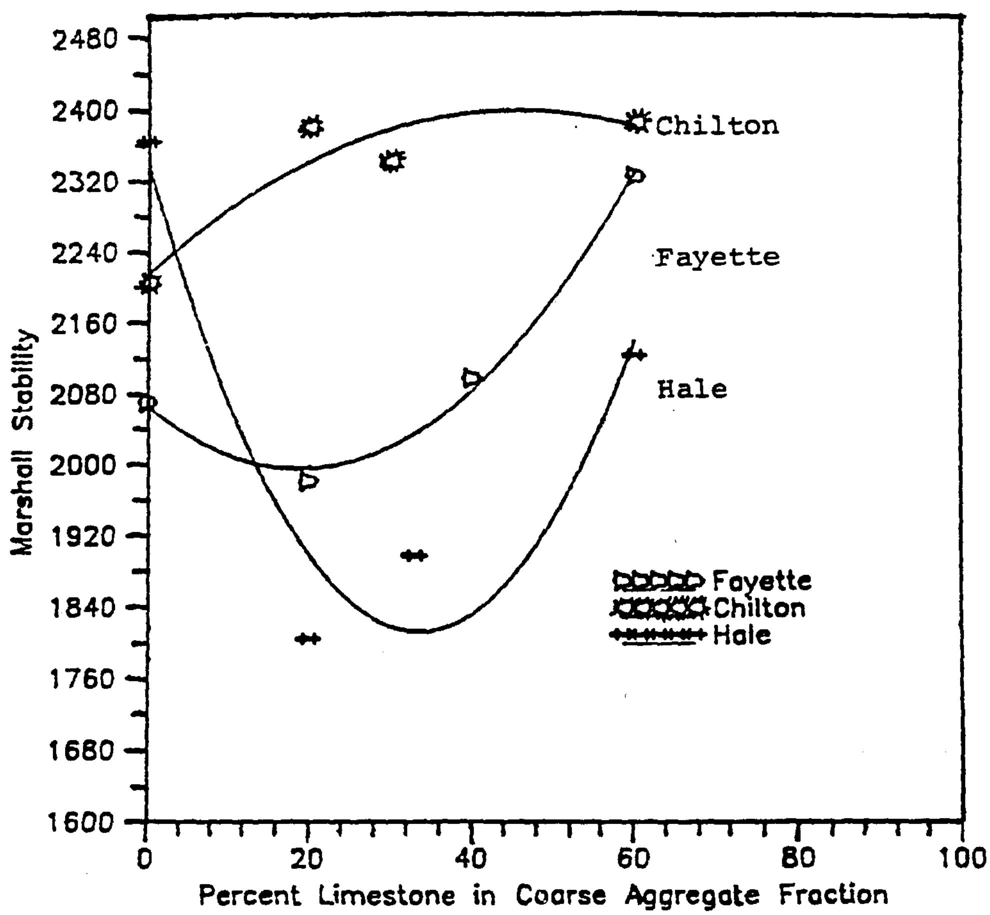

Figure 4. Marshall stability versus percent limestone in the coarse aggregate fraction.

Table 9.

Summary of Marshall mix design data for Fayette County

Control mix

% A.C.	Bulk S.G.	TMD	Air Voids	VMA	VFA	Stability	Flow
4.8	2.190	2.355	7.0	17.2	59.1	2425	9.3
5.2	2.213	2.340	5.4	17.0	68.1	2323	10.7
5.6	2.205	2.327	5.1	17.4	70.5	1951	10.3

20% Limestone mix

% A.C.	Bulk S.G.	TMD	Air Voids	VMA	VFA	Stability	Flow
4.8	2.216	2.378	6.8	17.3	60.6	2030	9.2
5.3	2.235	2.361	5.3	17.0	68.8	1970	9.1
5.8	2.248	2.345	4.1	16.9	75.7	1880	9.2

40% Limestone mix (experimental)

% A.C.	Bulk S.G.	TMD	Air Voids	VMA	VFA	Stability	Flow
4.7	2.231	2.393	6.8	17.0	60.3	2137	10.7
5.2	2.242	2.376	5.7	17.1	66.8	2039	10.0
5.7	2.242	2.360	5.0	17.5	71.7	2073	9.3
6.2	2.264	2.343	3.4	17.1	80.1	2018	10.3

60% Limestone mix

% A.C.	Bulk S.G.	TMD	Air Voids	VMA	VFA	Stability	Flow
4.4	2.235	2.412	7.4	17.0	56.5	2543	8.9
4.9	2.249	2.395	6.1	16.9	63.9	2226	8.7
5.4	2.267	2.378	4.6	16.7	72.2	2237	8.9

* TMD= Theoritical maximum density, VMA= Voids in mineral aggregate, VFA= Voids filled with asphalt.

for the 60% limestone mix is about 250 pounds larger than the control mix.

Chilton County: Marshall mix design data for Chilton County are summarized in Table 10. The air voids for the control and experimental mix were lower than those obtained by the AHD during mix design. It was decided to keep the same optimum asphalt content as the AHD, i.e. 5.2% for all the four mixes. Air voids at this asphalt content are about 3% for all four mixes.

Marshall stability values were estimated at 5.2% asphalt and plotted versus percent limestone coarse aggregate in Figure 4. The plot shows that the Marshall stability increased somewhat with increasing limestone content in the mix. Most of this increase occurred between the control and 20% mix with little if any increase indicated for higher limestone contents.

Table 10.

Summary of Marshall mix design data for Chilton County.

Control mix

% A.C.	Bulk S.G.	TMD	Air Voids	VMA	VFA	Stability	Flow
4.8	2.359	2.467	4.4	15.5	71.7	2169	8.8
5.2	2.378	2.452	3.0	15.2	80.2	2205	9.7
5.7	2.380	2.434	2.2	15.5	85.6	2041	10.7
6.2	2.376	2.417	1.7	16.1	89.4	1735	13.3

20% Limestone mix

% A.C.	Bulk S.G.	TMD	Air Voids	VMA	VFA	Stability	Flow
4.3	2.360	2.496	5.4	15.4	64.9	2671	9.7
4.8	2.385	2.477	3.7	14.9	75.2	2378	9.7
5.2	2.395	2.462	2.8	14.9	81.2	2245	11.0
5.7	2.402	2.444	1.7	15.1	88.7	2023	12.9

30% Limestone mix (Experimental)

% A.C.	Bulk S.G.	TMD	Air Voids	VMA	VFA	Stability	Flow
4.8	2.395	2.485	3.6	14.9	75.6	2671	10.5
5.2	2.393	2.470	3.1	15.3	79.7	2338	11.2
5.7	2.406	2.452	1.9	15.3	87.6	2307	12.0
6.2	2.400	2.433	1.4	15.5	91.2	1706	12.0

60% Limestone mix

% A.C.	Bulk S.G.	TMD	Air Voids	VMA	VFA	Stability	Flow
4.3	2.382	2.528	5.8	15.8	63.3	2370	9.9
4.8	2.409	2.509	4.0	15.3	73.9	2380	9.7
5.2	2.418	2.494	3.1	15.4	79.9	2209	11.8
5.7	2.421	2.475	2.2	15.7	86.0	2239	13.4

Hale County: Table 11 summarizes the Marshall mix design data for Hale County. Asphalt contents selected for the control and experimental mixes were the same as those obtained by the AHD during mix design. Asphalt contents giving the same air voids (4.5%) as that for the experimental mix were chosen for the 20% and 60% limestone mixes. However, the asphalt content selected for the control mix gave lower air voids (3%). The asphalt contents chosen for the mixes are as follows :

Control set	: 5.9%
20% limestone set	: 6.1%
33% limestone set (Experimental)	: 5.5%
60% limestone set	: 4.8%

Marshall stability values were estimated at the above asphalt contents and plotted in Figure 4. The shape of the curve is similar to that for Fayette County but the difference between the control and 20% limestone mix was more pronounced. This is likely due, in part, to the lower voids (3%) in the control mix. With consistent voids (4.5%) stability increased as percent limestone increased from 20 to 60%.

Summary: The effects of limestone on stability are not well established by this series of tests. This is partially due to inconsistencies in the composition (aggregate and asphalt content) and voids in the mixes. The mixes with limestone generally show that stability increases as percent limestone increases, but for lower limestone contents, structural benefits are clearly not demonstrated.

Table 11.

Summary of Marshall mix design data for Hale County

Control mix

% A.C.	Bulk S.G	TMD	Air Voids	VMA	VFA	Stability	Flow
5.4	2.330	2.448	4.8	17.0	71.9	2556	9.5
5.9	2.357	2.431	3.0	16.5	81.9	2364	10.3
6.4	2.361	2.413	2.2	16.8	87.1	2028	14.5

20% Limestone mix

% A.C.	Bulk S.G	TMD	Air Voids	VMA	VFA	Stability	Flow
5.1	2.293	2.469	7.1	18.5	61.6	1758	8.0
5.6	2.307	2.452	5.9	18.4	67.9	1780	7.8
6.1	2.317	2.434	4.8	18.5	74.1	1804	9.8

33% Limestone mix (Experimental)

% A.C.	Bulk S.G	TMD	Air Voids	VMA	VFA	Stability	Flow
4.9	2.333	2.489	6.3	17.3	63.9	1845	7.8
5.4	2.354	2.470	4.7	17.1	72.4	1897	9.0
5.9	2.363	2.453	3.6	17.2	78.9	1922	9.8

60% Limestone mix

% A.C.	Bulk S.G	TMD	Air Voids	VMA	VFA	Stability	Flow
4.5	2.384	2.530	5.8	16.1	64.0	2137	8.7
5.0	2.403	2.511	4.3	15.9	72.9	2112	8.7
5.5	2.416	2.492	3.1	15.9	80.5	2144	9.5
6.0	2.424	2.473	2.0	16.1	87.6	2224	12.5

* TMD= Theoritical maximum density, VMA= Voids in mineral aggregate, VFA= Voids filled with asphalt.

MODIFIED LOTTMAN TEST RESULTS

Results of the modified Lottman test are summarized in Table 12. Discussions of the results for each project are contained in the following paragraphs.

Fayette County: Figure 5 shows dry strength, wet strength and tensile strength ratio versus percent limestone coarse aggregate for Fayette County. Addition of limestone, somewhat, increased dry and wet strength. However, TSR decreased for 20 and 40% mixes and only increased for 60% mix. This indicates that limestone had little, if any, effect on the resistance of the mixes to moisture damage. However, it should be noted that the asphalt content of the control mix was 5.5% and the asphalt contents of the mixes with limestone were, respectively, 5.1, 5.2 and 5.0 for the 20, 40 and 60% limestone mixes. These smaller asphalt contents may have influenced the moisture resistance.

Chilton County: Results of the modified Lottman test are shown in Figure 6. Limestone appears to have reduced the dry strength of the mixes. However, wet strength and, thus, TSR increased as limestone percentage increased. The wet strength increased by about 60%. This indicates that the Chilton County limestone increased the mix resistance to moisture damage.

Hale County: Figure 7 shows that as the limestone content in the coarse aggregate fraction increased, the dry strength increased, but the wet strength decreased. As a result, TSR decreased. This indicates that addition of limestone in the coarse aggregate fraction decreased the mix

Table 12.

Summary of Modified Lottman Test results.

Fayette County

Mix	Dry Set Strength(psi)	Wet Set Strength(psi)	Tensile Strength Ratio
Control set	122.1	64.6	0.529
20% Limestone	151.4	77.5	0.512
40% Limestone (experimental)	128.7	63.7	0.495
60% Limestone	137.7	89.4	0.649

Chilton County

Mix	Dry Set Strength(psi)	Wet Set Strength(psi)	Tensile Strength Ratio
Control set	144.4	70.7	0.618
20% Limestone	119.8	96.1	0.802
30% Limestone (experimental)	119.5	96.4	0.807
60% Limestone	135.4	110.1	0.813

Hale County

Mix	Dry Set Strength(psi)	Wet Set Strength(psi)	Tensile Strength Ratio
Control set	83.3	59.5	0.714
20% Limestone	84.1	60.8	0.723
33% Limestone (experimental)	93.6	56.5	0.604
60% Limestone	96.0	51.7	0.538

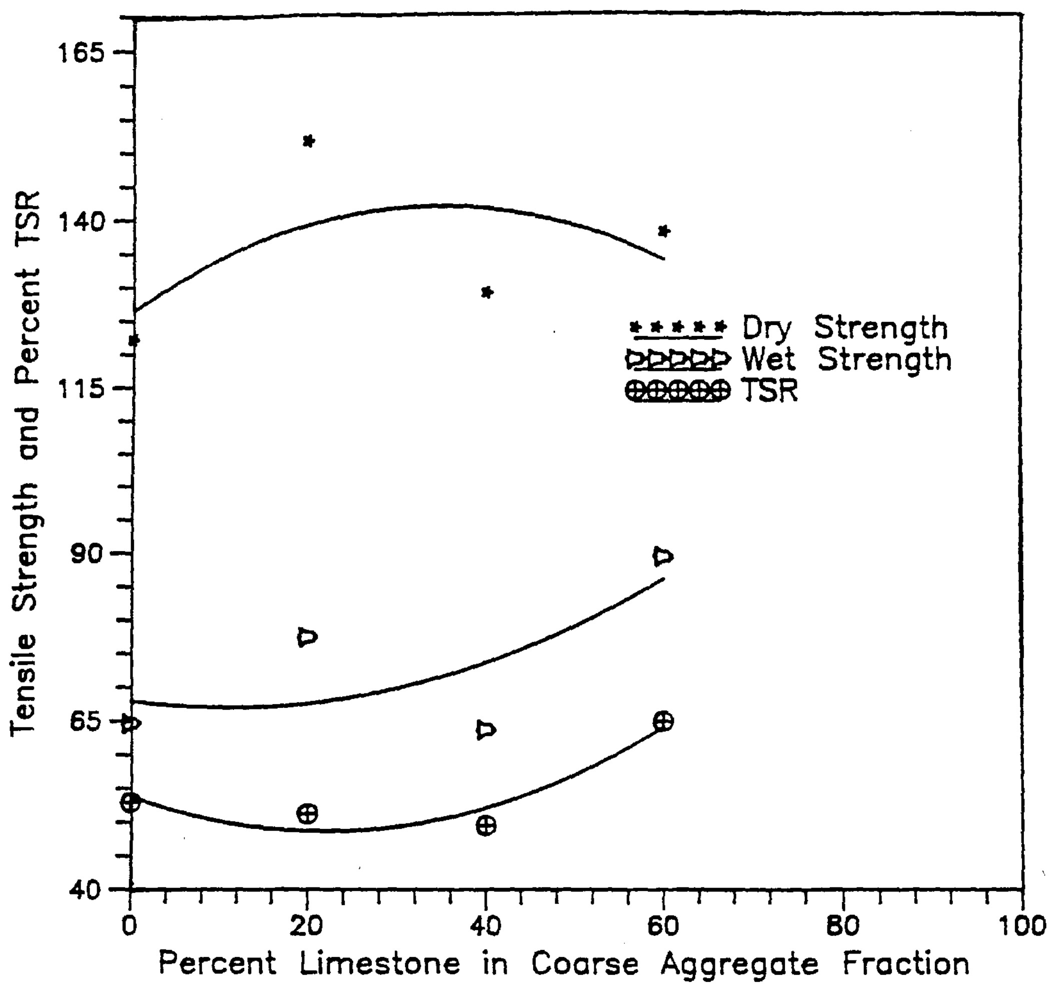

Figure 5. Percent limestone versus tensile strength and TSR (Fayette County).

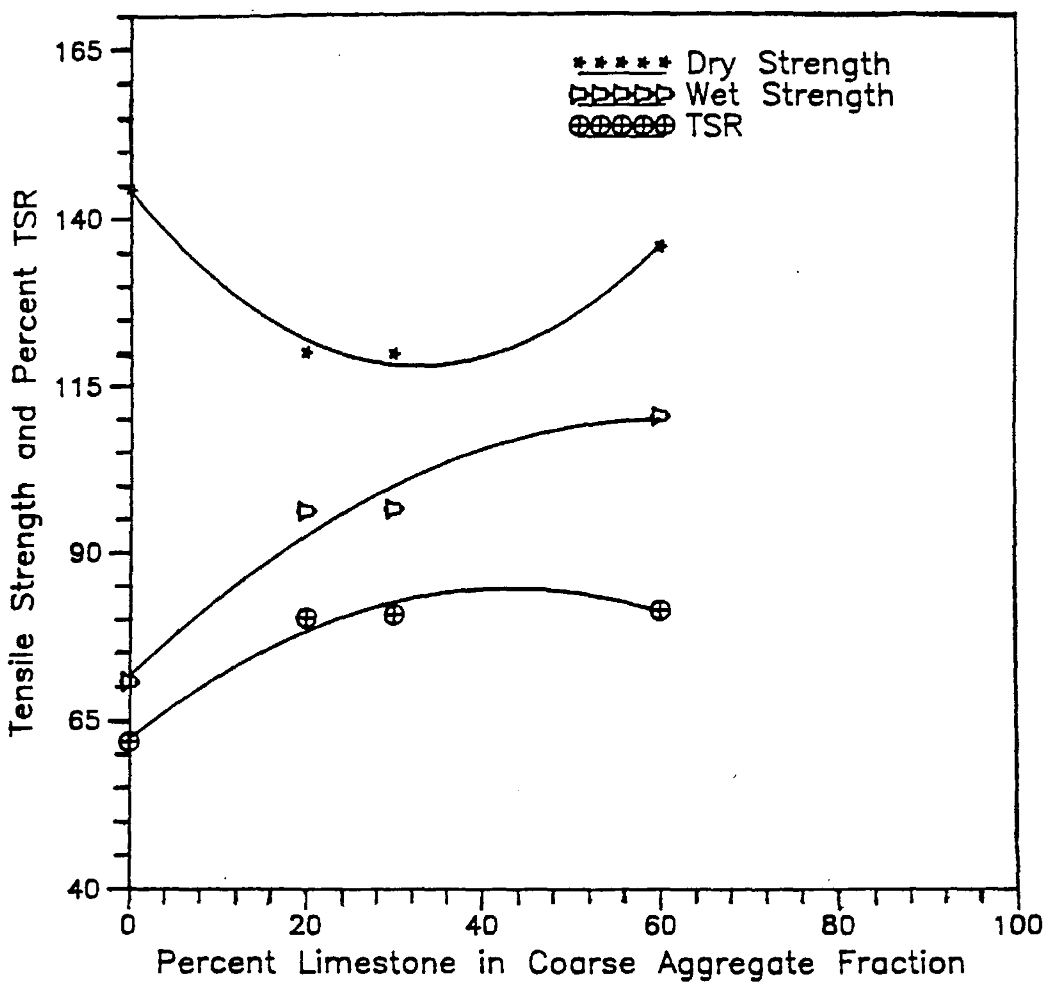

Figure 6. Percent limestone versus tensile strength and TSR (Chilton County).

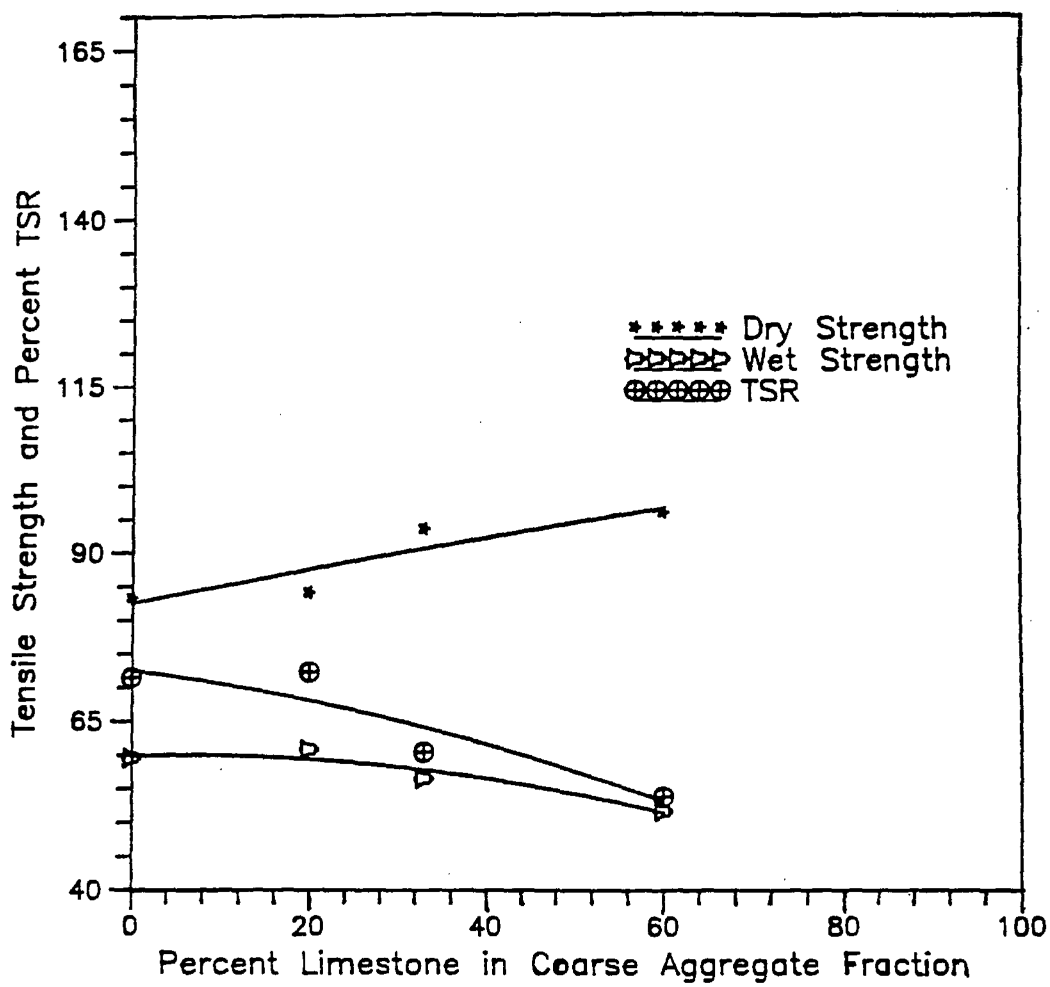

Figure 7. Percent limestone versus tensile strength and TSR (Hale County).

resistance to moisture damage. This trend is supported by field data obtained from ACIR-59-1(171) Sumter county project. The limestone used in that project had the same source (Dravo, Maylene) as that used in Hale County project. When the limestone content in Sumter County project was increased from 52% to 80% due to unavailability of RAP (recycled asphalt pavement), the percent TSR dropped from about 74% to around 45%. This drop was nearly equal to the increase in the limestone content and verifies the adverse effect of the particular source of limestone on the resistance of mix to moisture damage. An additional factor may have been the smaller asphalt contents (5.5 and 4.8%) for the 33 and 60% limestone mixes compared to the asphalt contents (5.9 and 6.1%) for the control and 20% limestone mixes.

<u>Summary</u>: The modified Lottman tests results are summarized in Figure 8. The plot shows that the Chilton County limestone improved the mix resistance to moisture damage, the Fayette County limestone seemed to have little or no effect, and the Hale County limestone seemed to have an adverse effect on the mix resistance to moisture damage. However, as noted earlier variable asphalt contents may also have affected the resistances of the mixes to moisture damage.

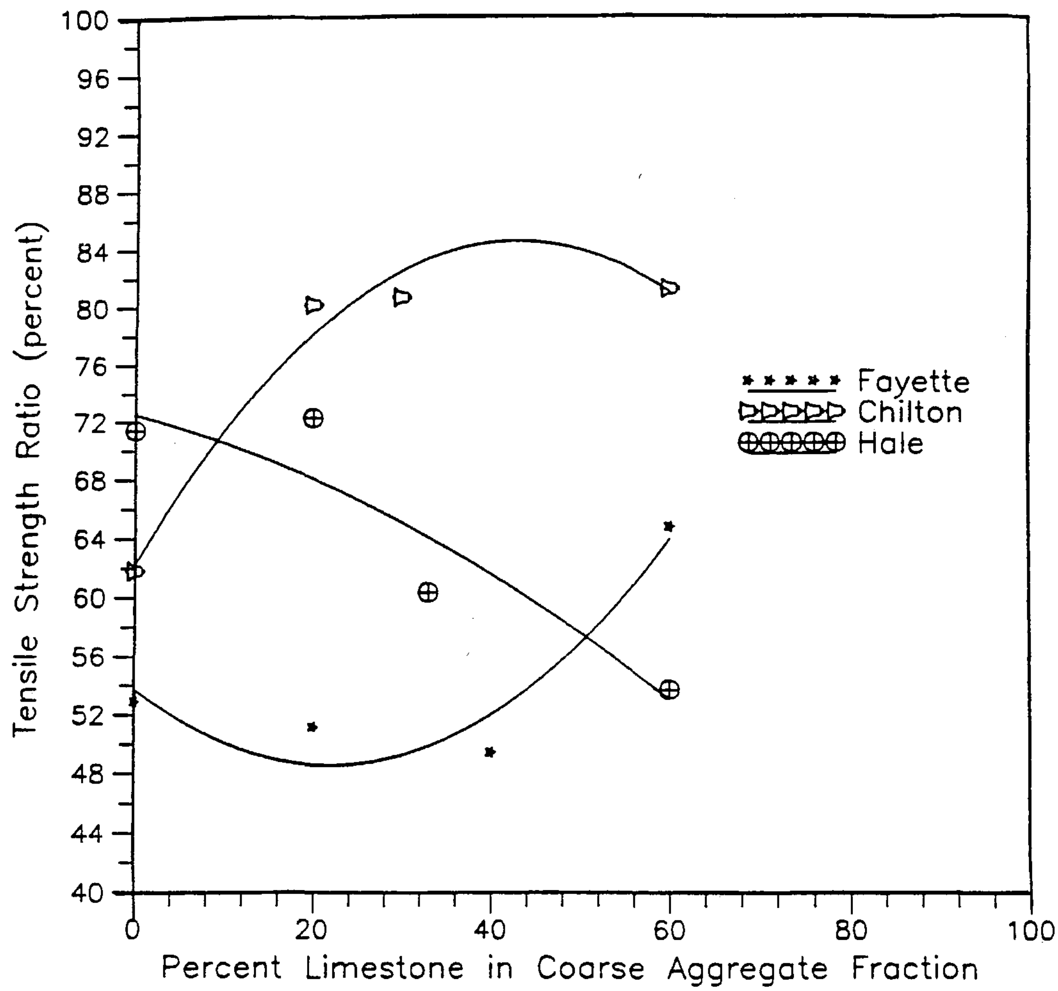

Figure 8. Summary of modified Lottman test results for all three counties.

POLISHING AND FRICTIONAL PROPERTIES OF AGGREGATES

Specimens composed of 100% crushed gravel and gravel-limestone blends from all projects were polished and BPN measured. About 70% of the samples tested survived the 9-hour polishing. Samples failed due to ravelling of the aggregates and cracking of the epoxy base. Failures were caused by repeated impact of the rubber tire as it moved across the uneven surface of the specimens. A majority of the failures occurred after 3 hours of polish. Due to the failure of some samples, the BPN reported for each set at various polish times is the average of 3 to 10 specimens.

Variations of BPN with polish time were observed to have a hyperbolic relation with time. This relationship has been observed in various other studies on polishing of bare aggregates (13). The hyperbolic relation is given below :

$$BPN = BPN_0 + \frac{t}{a+bt}$$

BPN = BPN at time, t.

BPN_0 = BPN at 0 polish hour (or original BPN).

where a and b are constants obtained using the following formulas :

$$a = \frac{t_1 t_2}{t_1 - t_2} \times \{ 1/dBPN_1 - 1/DBPN_2 \}$$

$$b = \frac{1}{t_1 - t_2} \times \{ 1/DBPN_1 - 1/dBPN_2 \}$$

where $dBPN_1 = BPN_1 - BPN_0$

$dBPN_2 = BPN_2 - BPN_0$

$dBPN_1$ and $DBPN_2$ are differential BPNs for polish times t_1 and t_2, respectively.

The hyperbolic relationship also gives the terminal (limiting) BPN or value of BPN at infinite time with the following expression :

$$BPN_L = BPN_0 + 1/b$$

where BPN_L is the limiting BPN value.

Test results of the polish and frictional testing are summarized in Table 13. The BPN values in Table 13 represent average of BPN readings obtained on individual polish specimens grouped according to percent limestone as follows:

Control set	:	0%
20% limestone set	:	15-25%
40% limestone set	:	25-45%
60% limestone set	:	45-80%

Table 13 also gives computed terminal (limiting) BPN values from the hyperbolic function model.

<u>Fayette County</u>: Polishing and frictional results for Fayette County are summarized in Figures 9 and 10. Figure 9 shows least square linear regression lines fits with BPN readings from individual polish samples for various polish hours. Figure 10 shows hyperbolic functions fit with average BPN readings from Table 19. As shown in Figures 9 and 10, all four blends experienced the largest reduction in BPN during the first 3 hours of polish. The slopes of the linear regression lines in Figure 9 show that BPN increased slightly as limestone percentage increased. The zero percent limestone line in Figure 10 is also below the other three lines at 9 hours polish. This indicates that the microtextural properties of the specimen

Table 13.

British pendulum numbers for testing of bare aggregates.

Fayette county

% Lmst	0 hrs polish	3 hrs polish	6 hrs polish	9 hrs polish	Terminal Values *
0	33	24	20	19	14
20	36	26	24	22	19
40	35	25	23	22	20
60	37	24	23	22	21

Chilton County

% Lmst	0 hrs polish	3 hrs polish	6 hrs polish	9 hrs polish	Terminal Values *
0	36	26	25	23	21
20	37	26	24	25	24
30	37	27	28	26	25
60	38	28	29	26	25

Hale County

% Lmst	0 hrs polish	3 hrs polish	6 hrs polish	9 hrs polish	Terminal Values *
0	38	29	28	27	26
20	38	27	25	23	20
33	37	28	26	26	25
60	38	26	25	22	19

* Terminal values obtained from Hypberbolic model.

** All BPN values other than terminal BPN values are average of 3-10 values

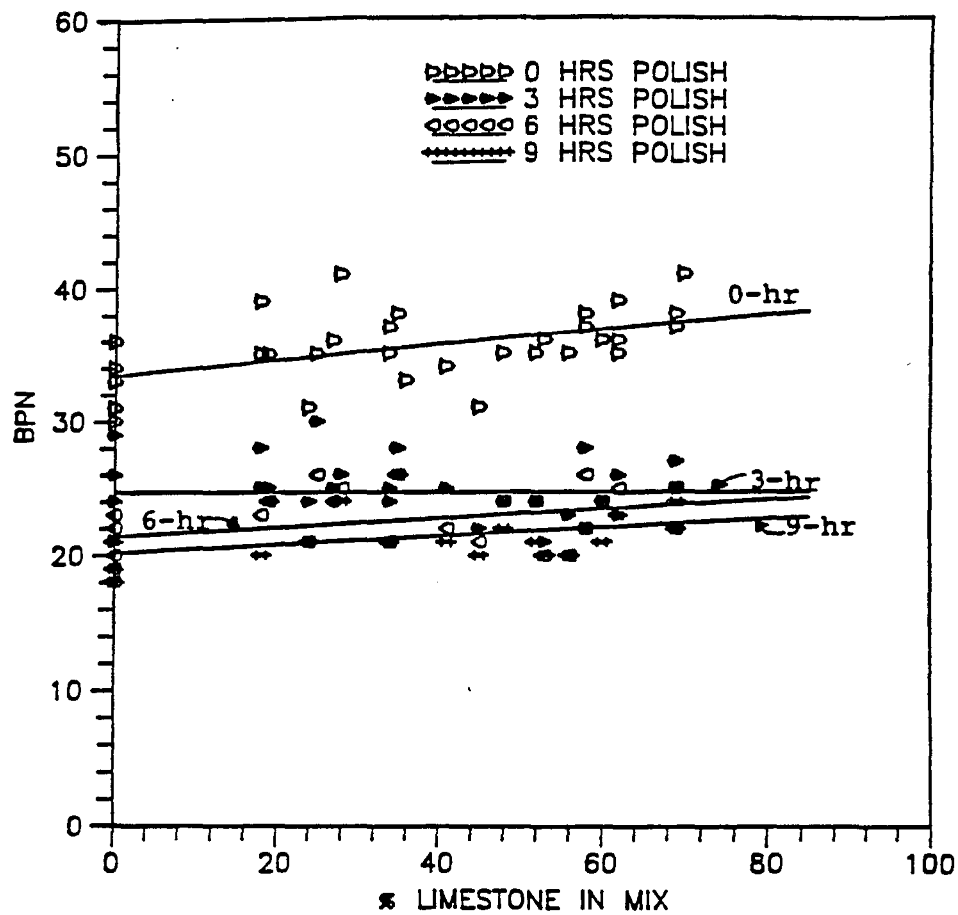

Figure 9. Limestone content versus BPN of aggregates (Fayette County).

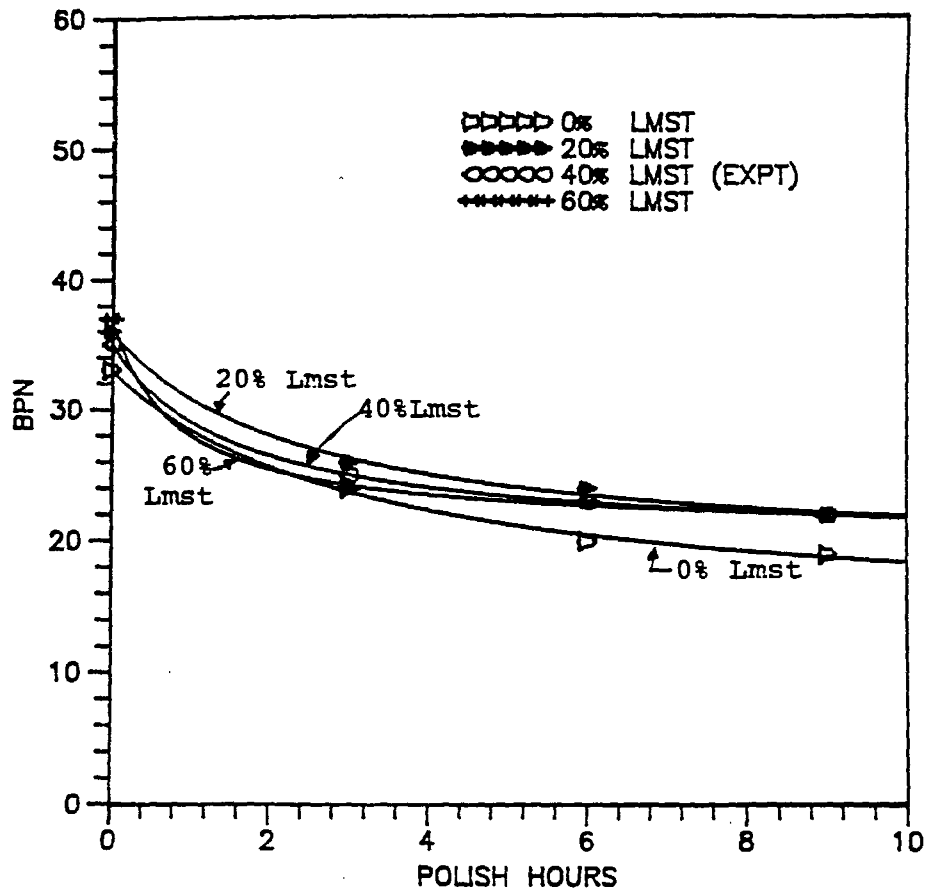

Figure 10. Polish hours versus BPN of aggregates (Fayette County).

improved as the limestone percentage in the specimen increased, although the improvement was quite small. Therefore, the limestone aggregate has slightly better polish resistance value than the gravel aggregate. An average R^2=0.99 was obtained for the hyperbolic fit curves shown in Figure 10.

Chilton County: Figure 11 and 12 show the results of tests on bare aggregates of Chilton County. As for Fayette county, large reductions in BPN was observed within the first 3 hours of polishing. The slopes of the linear regression lines in Figure 11 and the relationship of the hyperbolic functions at 9 hours polish in Figure 12 indicate slight improvement in polish resistance as the percent limestone increased. An average R^2=0.98 was obtained for the hyperbolic fit curves shown in Figure 12. On the basis of these test, the polish resistance of the limestone aggregate appears to be slightly better than the gravel aggregates.

Hale County: Figures 13 and 14 summarize the polishing data of Hale county aggregates. From Figure 13, it can be seen that the BPN readings before polishing remained at relatively same level when the limestone in the specimen was increased. However, after polishing the BPN values decreased as the limestone content in the specimen increased. The 20, 33 and 60% limestone lines after 9 hours polish are also lower than the control mix line in Figure 14. This indicates that this particular limestone is potentially more susceptible to polishing than the gravel aggregate. An average R^2=0.98 was obtained for the hyperbolic fit curves.

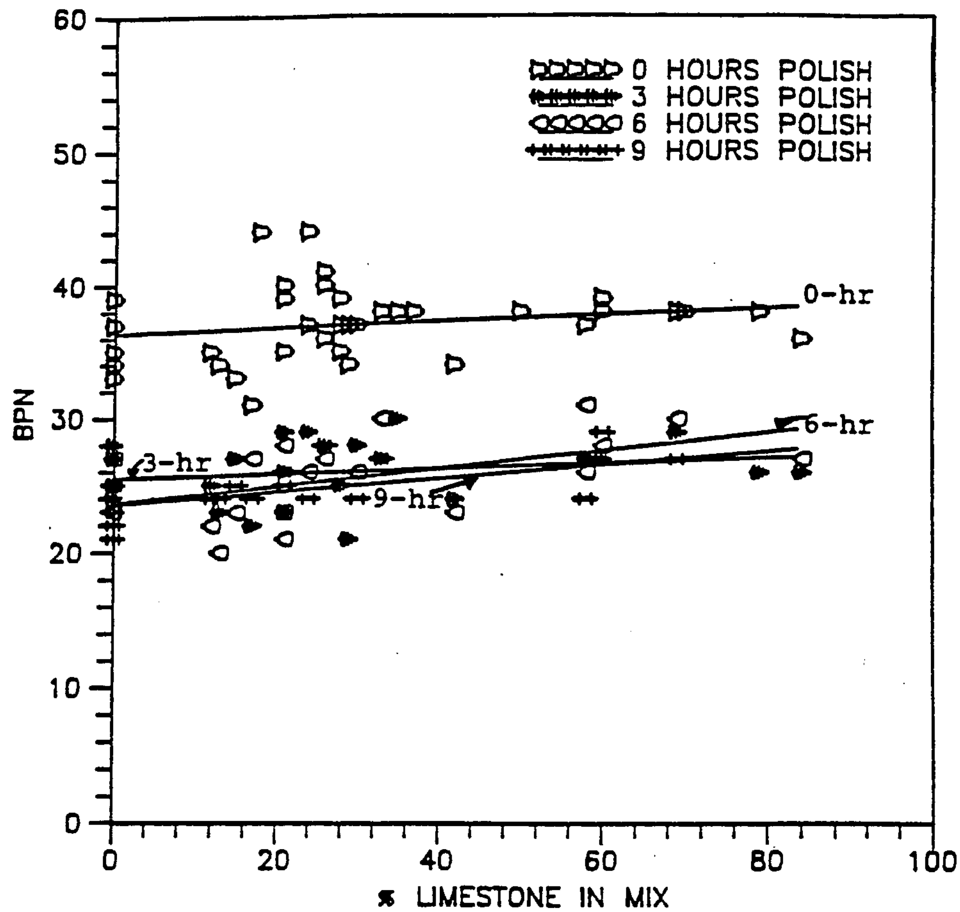

Figure 11. Limestone content versus BPN of aggregates (Chilton County).

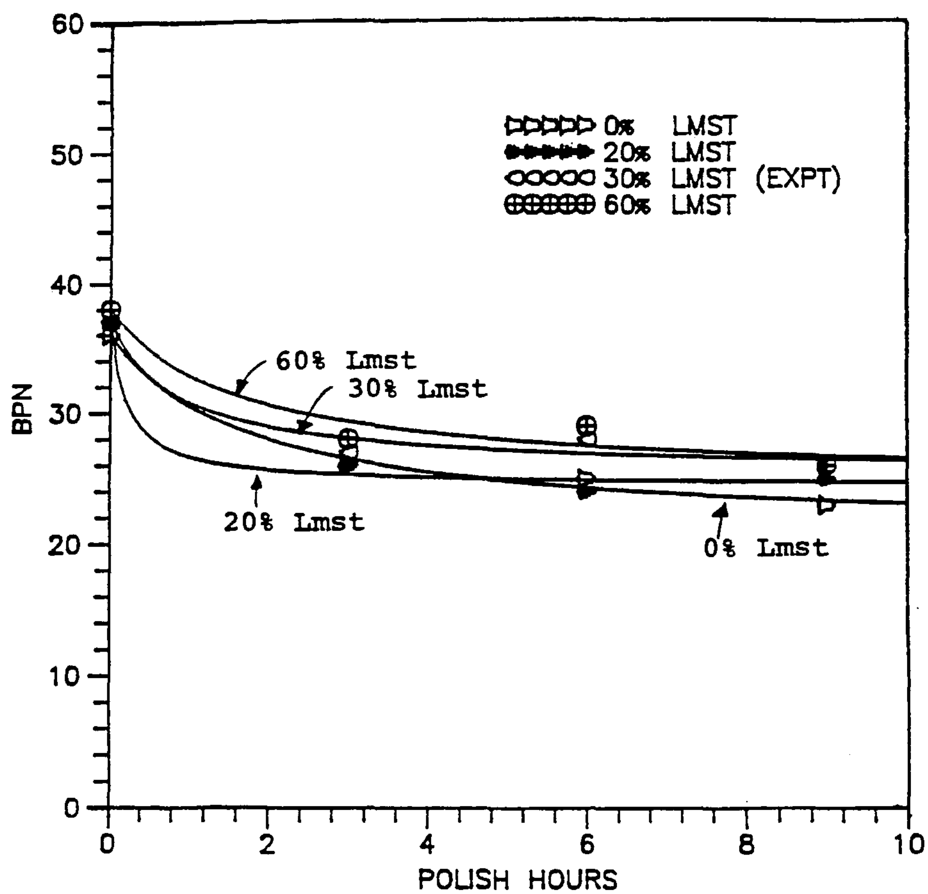

Figure 12. Polish hours versus BPN of aggregates (Chilton County).

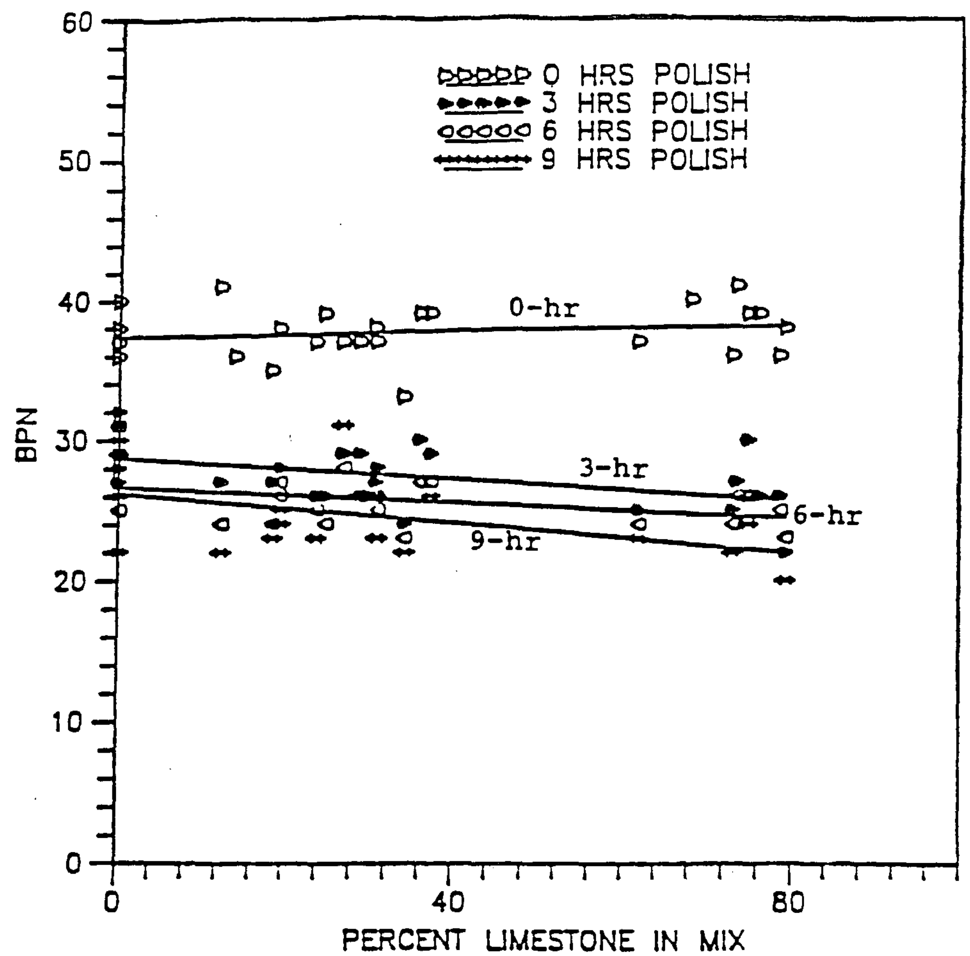

Figure 13. Limestone content versus BPN of aggregates (Hale County).

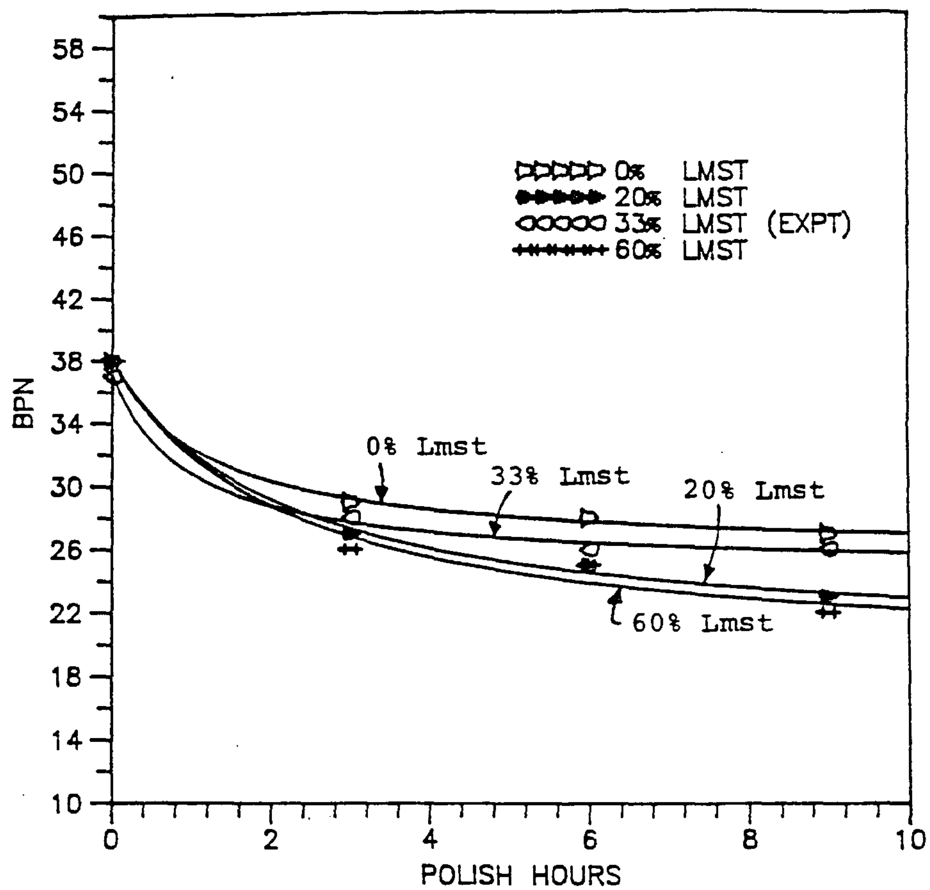

Figure 14. Polish hours versus BPN of aggregates (Hale County).

Summary: Based on the results discussed, it can be concluded that the Fayette and Chilton County limestones show slightly better polish resistance than the corresponding gravel aggregates. Polish tests on Hale County aggregates indicate that the initial friction properties of the limestone blends remained at about same level as the gravel aggregate specimens, but the limestone blends experienced loss in surface microtexture during polishing, resulting in decreased BPN readings. Substitution of this limestone for gravel could be detrimental to the frictional properties of a mix.

POLISHING AND FRICTIONAL PROPERTIES OF ASPHALT MIXES

Asphalt mixes of the control, experimental, 20% limestone and 60% limestone sets from Chilton and Hale Counties were tested on the British polish wheel and pendulum tester. Some samples were damaged on the polish wheel due to cracking of the surface and/or the epoxy base. The survivability rate of these specimens was 50-60 percent. This was significantly lower than that obtained for the bare aggregates. The process of bending rectangular asphalt mix slabs to conform to the mold shape could have caused some weakening of the specimens and could be one of the reasons for such low survivability rate.

The BPN values recorded for the top and bottom slices of the prepared specimens were compared by the "t-test". The statistical analysis showed that the difference between the BPN values of the top and bottom slices of the same specimen were insignificant with 95% confidence. This implies that the top or bottom surface can represent the mix and either or both can be tested. Testing of the unpolished middle slice of the Marshall specimen having both its sides sawed yielded low values because of polishing from sawing. Since the BPN readings of the middle slice were significantly lower than the 9-hour polish BPN of the top or bottom slices, testing of the middle sections was discontinued.

Results obtained from the polish testing of asphalt concrete mixes are shown in Table 14. Each data point is the average 3 to 10 specimens. Lease squares regression analyses were performed with this data to develop linear relationships between BPN and percent limestone (Figures 15 and 17), and to develop hyperbolic relationships between BPN and polish time (Figures 16 and 18).

<u>Chilton County</u>: The asphalt mix specimens for all four sets (control, experimental, 20% limestone and 60% limestone) experienced the largest reduction in BPN in the first 3 hours of polishing. This is similar to that experienced by the bare aggregates and is illustrated by the space between the lines in Figure 15 and the shape of the curves in Figure 16. The 60% limestone set showed slightly higher BPN than the other sets before polishing but after polishing the linear regression lines in Figure 15 become almost flat indicating that the BPN did not change significantly as the percent limestone in the coarse aggregate fraction increased. In Figure 16 the hyperbolic curves converge as polish time increases. Both indicate that the increasing limestone content in the coarse aggregate portion of the mix did not affect the frictional properties of the mix. Tests on the bare coarse aggregates indicated that increasing limestone content slightly improved frictional properties.

Table 14.

British pendulum numbers of asphalt concrete mixes.

Chilton County

% Lmst	0 hrs polish	3 hrs polish	6 hrs polish	9 hrs polish
0	46	39	35	33
20	46	37	32	32
30	45	35	33	32
60	48	38	34	33

Hale County

% Lmst	0 hrs polish	3 hrs polish	6 hrs polish	9 hrs polish
0	47	40	38	35
20	48	41	39	38
33	45	39	38	37
60	49	42	38	37

** All BPN values reported are average of 3-10 readings.

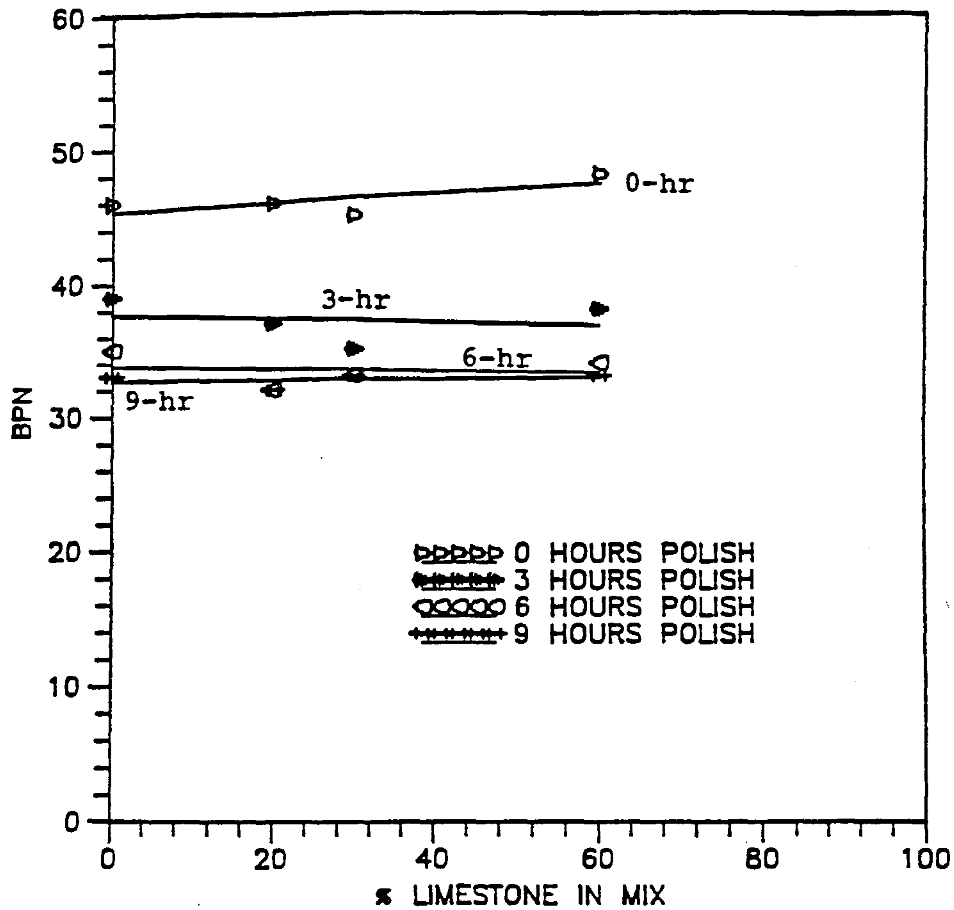

Figure 15. Limestone content versus BPN of asphalt mixes (Chilton County).

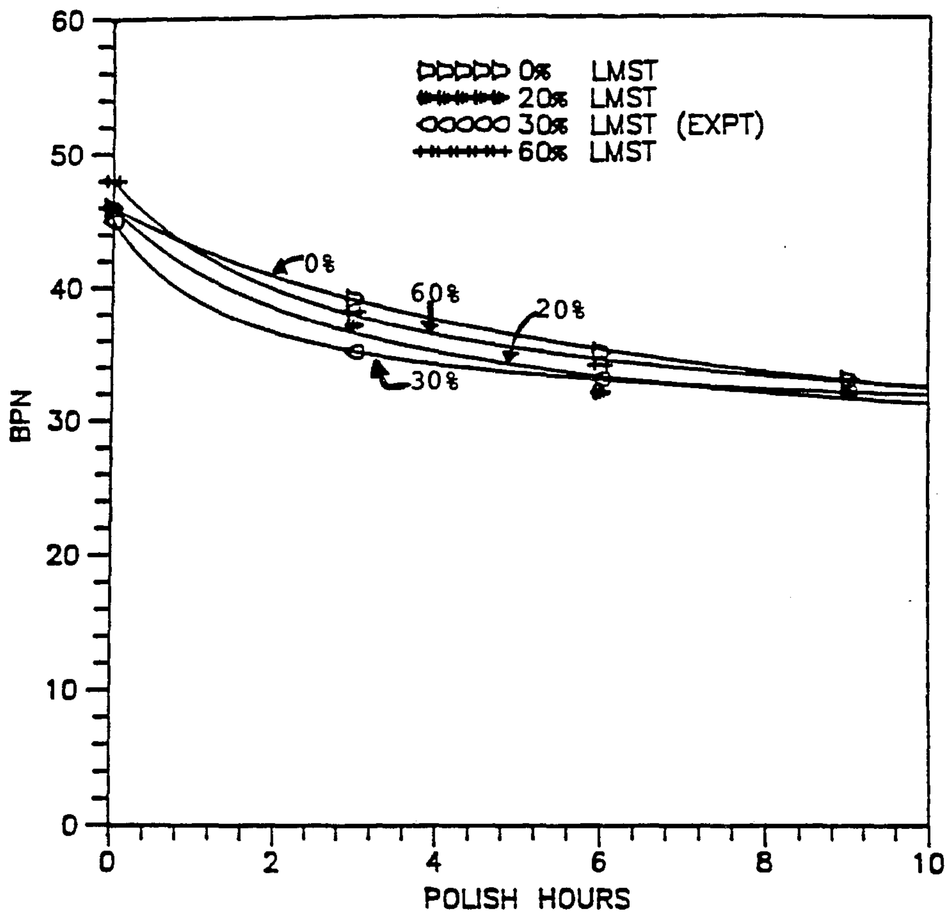

Figure 16. Polish hours versus BPN of asphalt mixes (Chilton County).

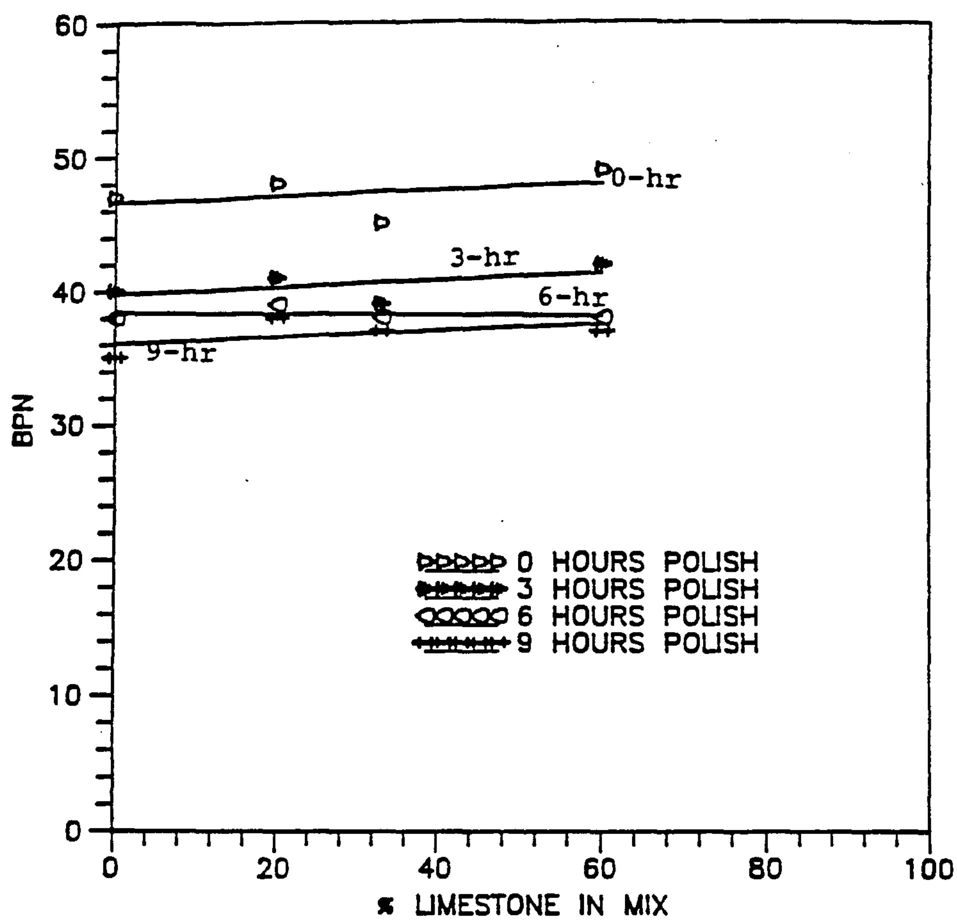

Figure 17. Limestone content versus BPN of asphalt mixes (Hale County).

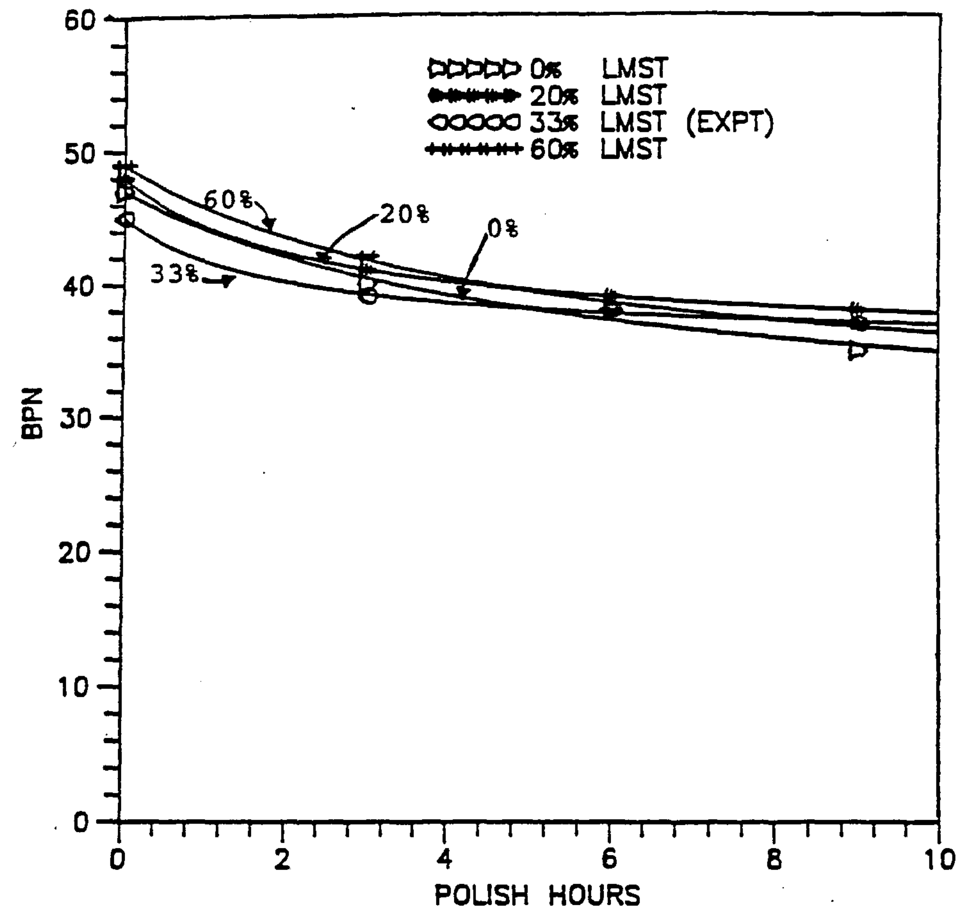

Figure 18. Polish hours versus BPN of asphalt mixes (Hale County).

Hale County: The 60% limestone mix showed slightly higher BPN than the other sets before polishing and maintained nearly the same trend with polishing (Figures 17 and 18). This is contrary to the results of polish resistance of bare aggregates of Hale county (Figures 13 and 14) which indicate that limestone decreased frictional properties.

COMPARISON OF AGGREGATE AND MIX BPN'S

Correlation between the BPN of the bare aggregates and asphalt mixes of Chilton and Hale Counties showed good overall correlation (Figure 19) with $R^2 = 0.828$. The correlation is surprisingly strong and its implications are unclear. Additional data with a wider range of coarse and fine aggregates, gradations and mix designs are needed for clarification. Since mix frictional and polish characteristics are influenced by multiple factors, it is anticipated that additional data may weaken the correlation although, some relationship with coarse aggregate properties will remain.

For Chilton County, the trends for 9 hours polish exhibited in Figures 11 and 15 are that BPN of the coarse aggregates increases and that BPN of the mix remains almost constant as limestone percentage increases. For Hale County, the trends for 9 hours exhibited in Figures 13 and 17 are that aggregate BPN decreases and that mix BPN increases as limestone percentage increases. Although the trends tend to send conflicting signals regarding the effect of limestone coarse aggregate, they in general fail to implicate limestone for loss of skid resistance.

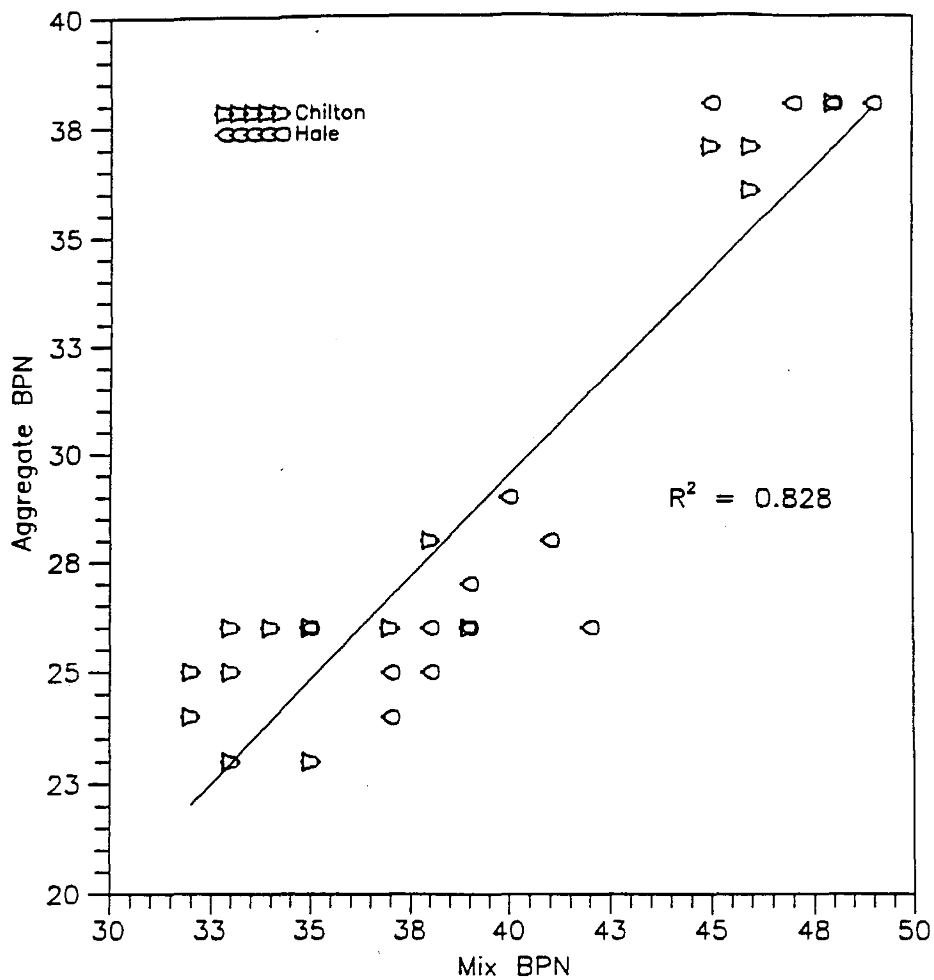

Figure 19. Mix BPN versus aggregate BPN.

RESULTS OF IN-SITU SKID TESTING

Four sets of field frictional resistance measurements consisting of BPN and SN values are reported in this section. The first set of measurements was recorded in January 1990. The Hale County project was not constructed at this time, hence skid measurements were only taken in Fayette and Chilton Counties. Additional measurements were made in August 1990, January 1991 and May 1991. Also included are SN values taken in Fayette and Chilton counties during 1988 soon after the pavements were constructed.

January 1990 Tests: Results of the first set of readings are summarized in Table 15. The BPN readings obtained on Fayette County show that the experimental section had higher microtexture levels. The friction levels, as measured by the SN readings, were nearly the same for the control and experimental sections. This indicates that experimental section in Fayette County was performing equal to or slightly better than the control section.

Based on the BPN readings taken in Chilton County (Table 15), higher levels of microtexture were observed on the SBL of the experimental section compared to SBL of the control section. The microtexture level on the NBL of the experimental section was nearly the same as the control section. The SN values indicate the skid resistance of the Chilton county experimental section was higher than that of the control section.

A linear least square regression equation was fit to the data relating BPN to SN. A coefficient of determination, $r^2 = 0.16$, was obtained indicating poor correlation. This was not unexpected since the

Table 15

Summary of SN and BPN numbers recorded on January 1990.

Fayette County (SR 13)

			BPN			SN		
	Age in months	Estmate Traffic x 10^6	NBL	SBL	Avg	NBL	SBL	Avg
Control	22	0.36	44	42	43	44	42	43
40% Lmst (expt)	22	0.36	46	44	45	43	42	43

Chilton County (US 31)

			BPN			SN		
	Age in months	Estimate Traffic x 10^6	NBL	SBL	Avg	NBL	SBL	Avg
Control	27	1.54	55	53	54	46	43	45
30% Lmst (expt)	27	1.54	54	60	57	49	46	48

skid trailer measures both microtexture and macrotexture at 40 mph while the British pendulum measures primarily microtexture at 4-5 mph.

August 1990 Tests: The second set of observations included the Hale County project completed in July 1990. Results of the second set of observations are summarized in Table 16. A coefficient of determination, $R^2 = 0.33$, was obtained from a least square linear regression again indicating poor correlation between BPN and SN.

In Fayette County, microtexture levels of the NBL of control section were higher but the control section of the SBL had same BPN readings as the experimental section. The SNs on both the lanes for both the mixes were the same. These values are similar to those obtained for the January 1990 measurements and indicate very little difference in the frictional properties of the control and experimental sections.

For Chilton County the BPN readings for the SBL experimental section was the same as that of the control section but the NBL of the experimental section showed higher microtexture levels. The SN readings also showed higher friction levels on both lanes of the experimental section compared to the control section. These values are relatively the same as those obtained for the January 1990 measurement and generally indicate higher friction in the experimental section.

For Hale County, BPN and SN readings were higher for the experimental section, indicating that the initial microtexture and macrotexture levels on the experimental section were higher than those of the control section.

Table 16

Summary of BPN and SN recorded in August 1990

Fayette County (SR 13)

			BPN			SN		
	Age in months	Estmate Traffic x 10^6	NBL	SBL	Avg	NBL	SBL	Avg
Control	29	0.48	41	45	43	45	44	45
40% Lmst (expt)	29	0.48	37	45	41	45	44	45

Chilton County (US 31)

			BPN			SN		
	Age in months	Estimate Traffic x 10^6	NBL	SBL	Avg	NBL	SBL	Avg
Control	34	1.94	38	42	40	41	37	39
30% Lmst (expt)	34	1.94	42	42	42	46	45	46

Hale County (SR 25)

			BPN			SN		
	Age in months	Estimate Traffic x 10^6	NBL	SBL	Avg	NBL	SBL	Avg
Control	2	0.03	51	52	52	50	50	50
30% Lmst (expt)	2	0.03	56	54	55	54	51	53

January 1991 Tests: The third set of measurements are tabulated in Table 17. The BPN and SN values on the experimental sections of Fayette and Hale Counties are nearly the same as the respective control sections. Chilton County values show that the microtexture and macrotexture levels on the experimental section are higher than those on the control section. The coefficient of determinations R^2, for the relationship between BPN and SN was 0.11.

May 1991 Tests: The fourth set of measurements are tabulated in Table 18. Only the SN values on the experimental section of Chilton County were observed to be higher than the control section. SN and BPN readings on experimental sections of Fayette and Hale Counties, and BPN readings on the experimental section of Chilton County were about the same as the respective control sections. An R^2 value of 0.02 was obtained for the linear regression relationship between BPN and SN indicating no correlation.

Summary Analyses: Statistical analyses ("t-test") to determine the significance of differences between control and experimental section readings (SN and BPN) were conducted for all three counties with 95% confidence level. Results from these analyses are summarized in Table 19. The computed "t-statistics" are consistently larger than tabular values (in parentheses) only for SN values for Chilton County. This indicates that only SN values for control and experimental sections of Chilton County are significantly different. Numerically SN values for the Chilton County experimental section were larger than the control section implying statistically significant higher frictional resistance.

Table 17

Summary of BPN and SN recorded in January 1991.

Fayette County (SR 13)

			BPN			SN		
	Age in months	Estmate Traffic x 10^6	NBL	SBL	Avg	NBL	SBL	Avg
Control	34	0.56	48	46	47	43	42	43
40% Lmst (expt)	34	0.56	48	48	48	42	42	42

Chilton County (US 31)

			BPN			SN		
	Age in months	Estimate Traffic x 10^6	NBL	SBL	Avg	NBL	SBL	Avg
Control	39	2.22	45	42	44	42	40	41
30% Lmst (expt)	39	2.22	49	47	48	46	46	46

Hale County (SR 25)

			BPN			SN		
	Age in months	Estimate Traffic x 10^6	NBL	SBL	Avg	NBL	SBL	Avg
Control	7	0.10	49	48	49	51	52	52
30% Lmst (expt)	7	0.10	52	47	50	52	52	52

Table 18

Summary of BPN and SN recorded in May 1991.

Fayette County (SR 13)

			BPN			SN		
	Age in months	Estmate Traffic x 10^6	NBL	SBL	Avg	NBL	SBL	Avg
Control	38	0.63	44	46	45	41	41	41
40% Lmst (expt)	38	0.63	45	42	44	41	41	41

Chilton County (US 31)

			BPN			SN		
	Age in months	Estimate Traffic x 10^6	NBL	SBL	Avg	NBL	SBL	Avg
Control	43	2.45	43	42	43	40	37	39
30% Lmst (expt)	43	2.45	43	45	44	44	44	44

Hale County (SR 25)

			BPN			SN		
	Age in months	Estimate Traffic x 10^6	NBL	SBL	Avg	NBL	SBL	Avg
Control	11	0.16	45	42	42	46	47	47
30% Lmst (expt)	11	0.16	45	43	44	47	47	47

Table 19

Statistical comparison of control and experimental friction

Fayette County

Test Date	BPN		SN	
	NBL	SBL	NBL	SBL
1/90	1.07(2.09)	0.79(2.09)	0.44(2.09)	0.14(2.09)
8/90	1.96(2.07)	0.10(2.07)	0.54(2.09)	0.03(2.09)
1/91	0.01(2.09)	1.76(2.09)	0.88(2.09)	0.05(2.09)
5/91	0.42(2.09)	1.48(2.09)	0.04(2.09)	0.01(2.09)

Chilton County

Test Date	BPN		SN	
	NBL	SBL	NBL	SBL
1/90	0.24(2.20)	2.17(2.20)	2.74(2.20)	2.87(2.20)
8/90	2.71(2.26)	0.19(2.26)	3.94(2.26)	5.74(2.26)
1/91	1.71(2.36)	2.71(2.36)	5.43(2.31)	7.08(2.31)
5/91	0.03(2.31)	0.39(2.31)	1.57(2.26)	6.86(2.26)

Hale County

Test Date	BPN		SN	
	NBL	SBL	NBL	SBL
1/90	-	-	-	-
8/90	2.22(2.09)	0.51(2.09)	4.53(2.11)	0.15(2.11)
1/91	2.13(2.11)	0.71(2.11)	2.09(2.12)	0.08(2.12)
5/91	0.03(2.12)	0.10(2.12)	0.34(2.11)	0.03(2.11)

Number of observations vary from 5 to 17. t-values obtained from t-distribution curve (95% confidence interval) for given number of observations are shown in parentheses.

When "t-statistics" are lower than tabular values, differences between experimental and control SN and BPN values are not significantly different at a 95% confidence level. This is most often the case in Table 19, except as noted above. The five other instances where the "t-test" indicates significant differences (2-BPN for Chilton County, 2-BPN for Hale County and 1-SN for Hale County), values on the experimental section are larger implying higher frictional resistance.

Correlations between SN and BPN for the four sets of readings were poor. To see if these poor correlations were due to the diversity of data from three counties, all the data from each county were combined and linear regressions performed. The following coefficients of determination were computed:

Fayette County, $R^2 = 0.00$
Chilton County, $R^2 = 0.18$
Hale County, $R^2 = 0.24$

Again indicating essentially no correlation between SN and BPN is indicated. Finally all data from the three counties and the four testing times were combined and a regression analysis performed. A coefficient of determination, $R^2 = 0.16$, was obtained strengthening the contention that BPN and SN are measures different frictional characteristics.

The data from the four sets of measurements (Tables 15-18) were combined with the early skid trailer data in Table 20 to produce a series of plots of SN and BPN versus pavement surface age and applied traffic.

Table 20

Summary of skid data (SN) obtained from the AHD.

Chilton County (US 31)

Date of test	Age in mths	Estm. Traf $\times 10^6$	Control section		Expt. section (30% Limestone)	
			NBL	SBL	NBL	SBL
1/88	3	0.17	43	43	47	46
3/88	5	0.28	38	38	45	38
9/88	11	0.63	35	34	40	37

Fayette County (SR 13)

Date of test	Age in mths	Estm. Traf $\times 10^6$	Control section		Expt. section (40% Limestone)	
			NBL	SBL	NBL	SBL
6/88	3	0.05	48	46	43	44

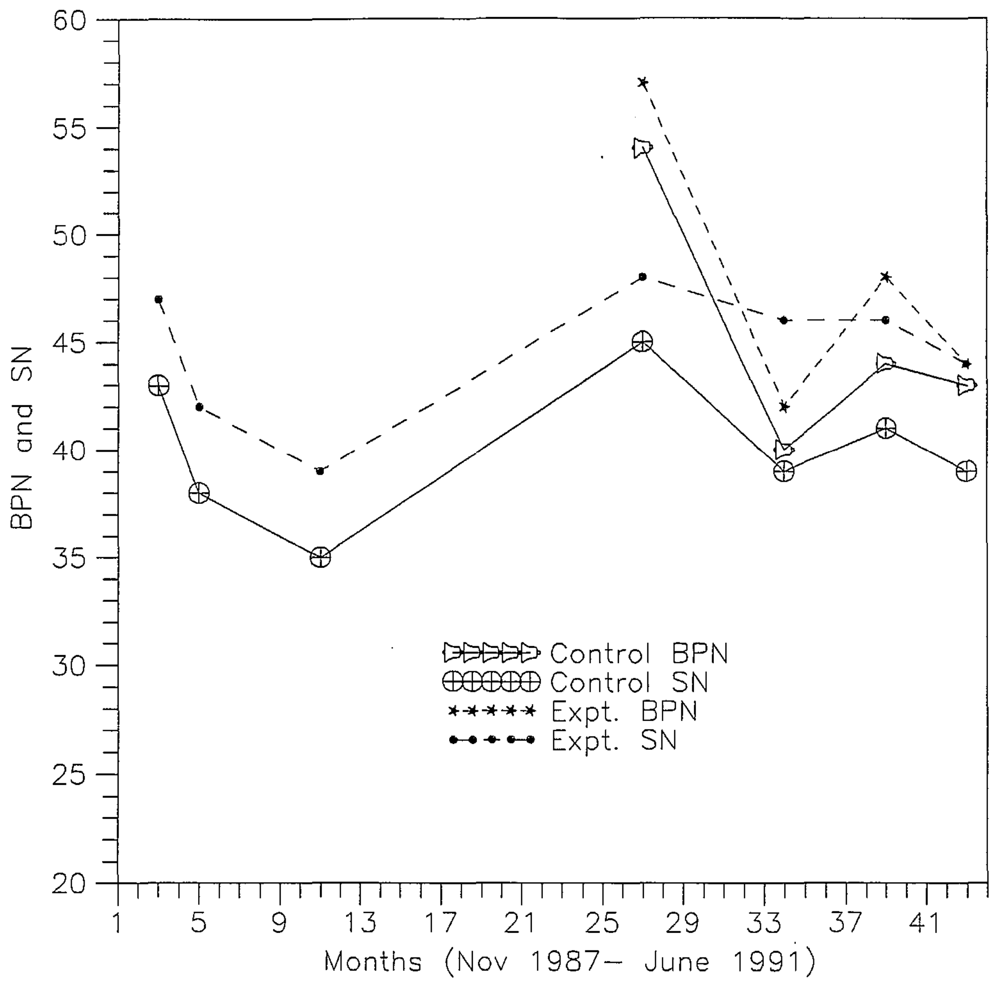

Figure 20. Variation of in-situ BPN and SN with time for Chilton County.

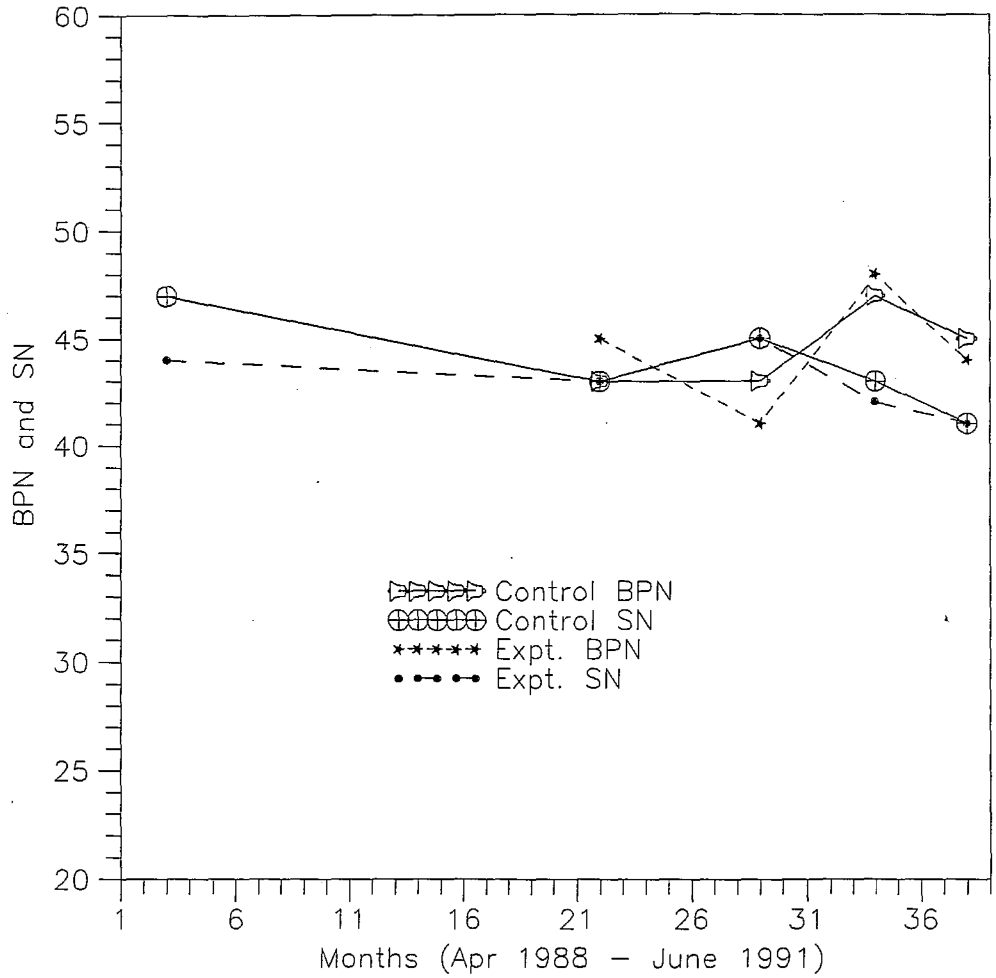

Figure 21. Variation of in-situ BPN and SN with time for Fayette County.

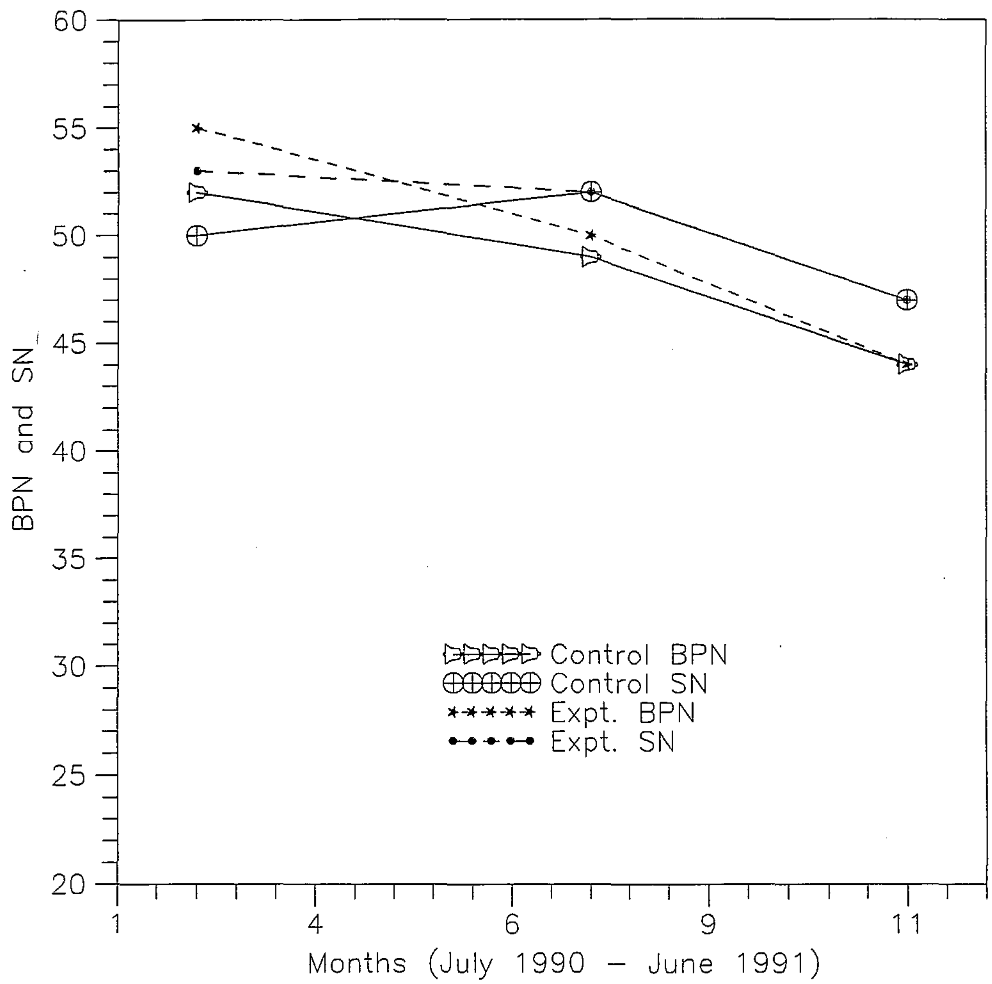

Figure 22. Variation of in-situ BPN and SN with time for Hale County.

Figures 20, 21 and 22 show the variation of SN and BPN with time (months) for Chilton, Fayette and Hale Counties, respectively. Figures 20 and 21 show evidence of seasonal variability but no general downward trend that would indicate loss of frictional resistance. This cyclical pattern is thought to be due to rainfall, temperature and weather related factors (14). Figure 22, for Hale County, shows no signs of seasonal variations but does exhibit a general downward trend indicating loss of frictional resistance.

Figures 23, 24 and 25 show the variation of SN and BPN with accumulated millions of vehicle passes for Chilton, Fayette and Hale County, respectively and demonstrate the same trends as Figures 20, 21 and 22. The SNs on most of the asphalt concrete pavements in Alabama using crushed aggregates have been found to level off after about 3 million vehicle passes (43). This means that the pavements in Chilton County should have almost reached minimum friction resistance. Other researchers have reported similar trends or that the decrease in SN becomes insignificant (36,14). Some surfaces have also shown increase in SN due to surface renewal and/or surface rejuvenation as surface wears occurs (35,14). The SN recordings in the future will give a better understanding of the pavement performance, but it appears that sufficient traffic has been applied to conclude that the experimental mix provides better skid resistance than the control mix for Chilton County. Additional traffic will be required before definitive conclusions can be drawn for Fayette and Hale Counties, although, initial data suggest almost equal performance for the control and experimental sections.

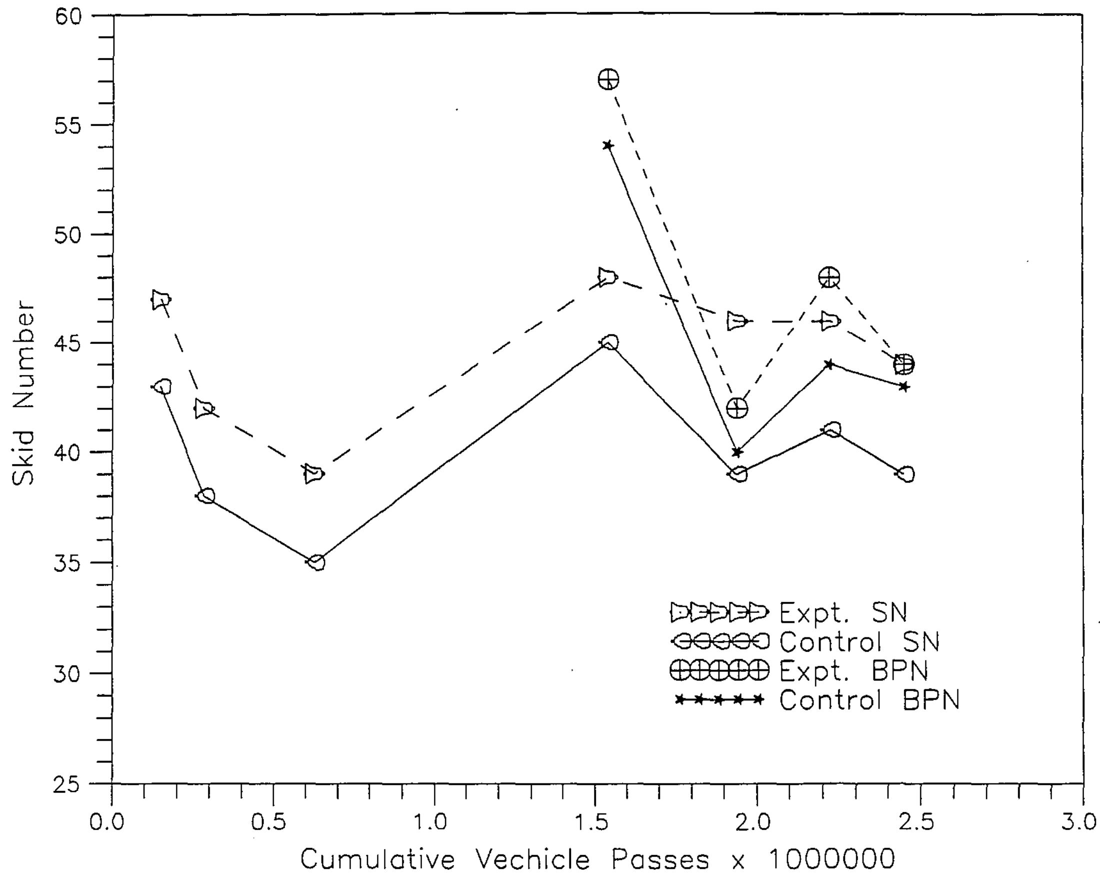

Figure 23. SN versus cumulative vehicle passes (Chilton County).

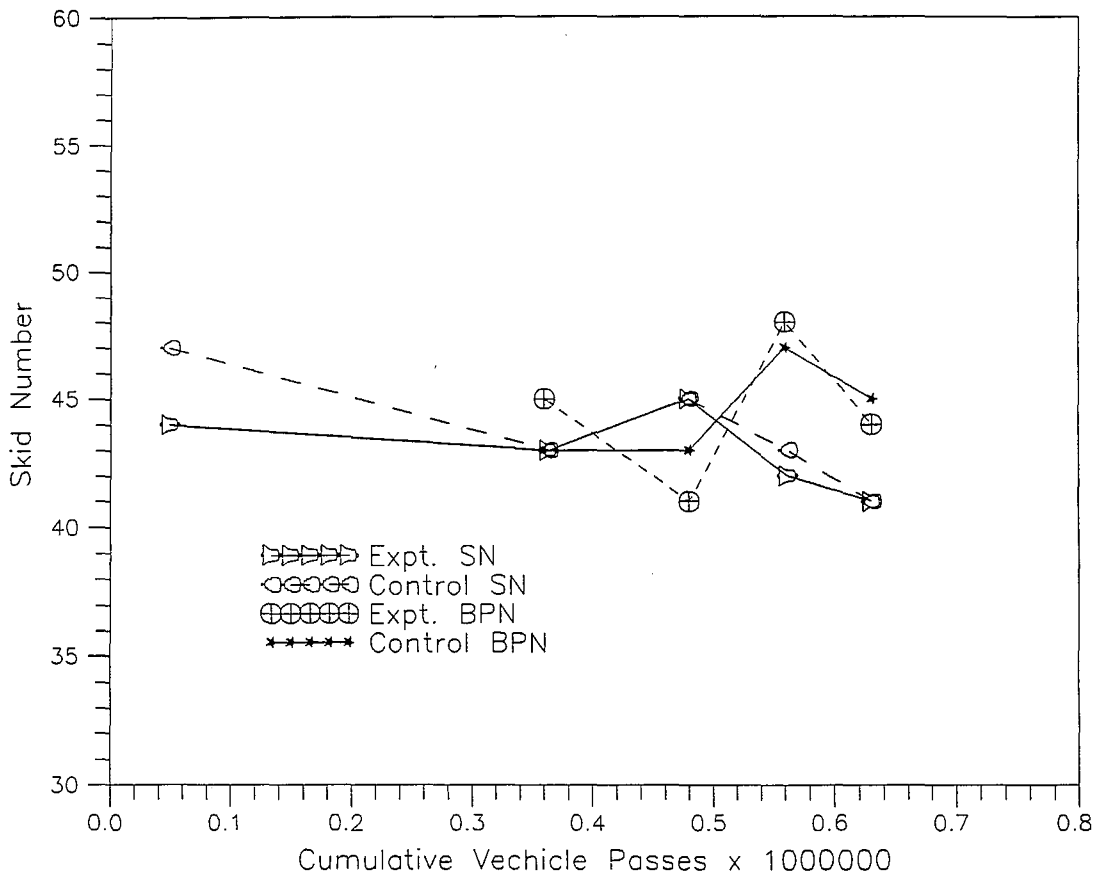

Figure 24. SN versus cumulative vehicle passes (Fayette County).

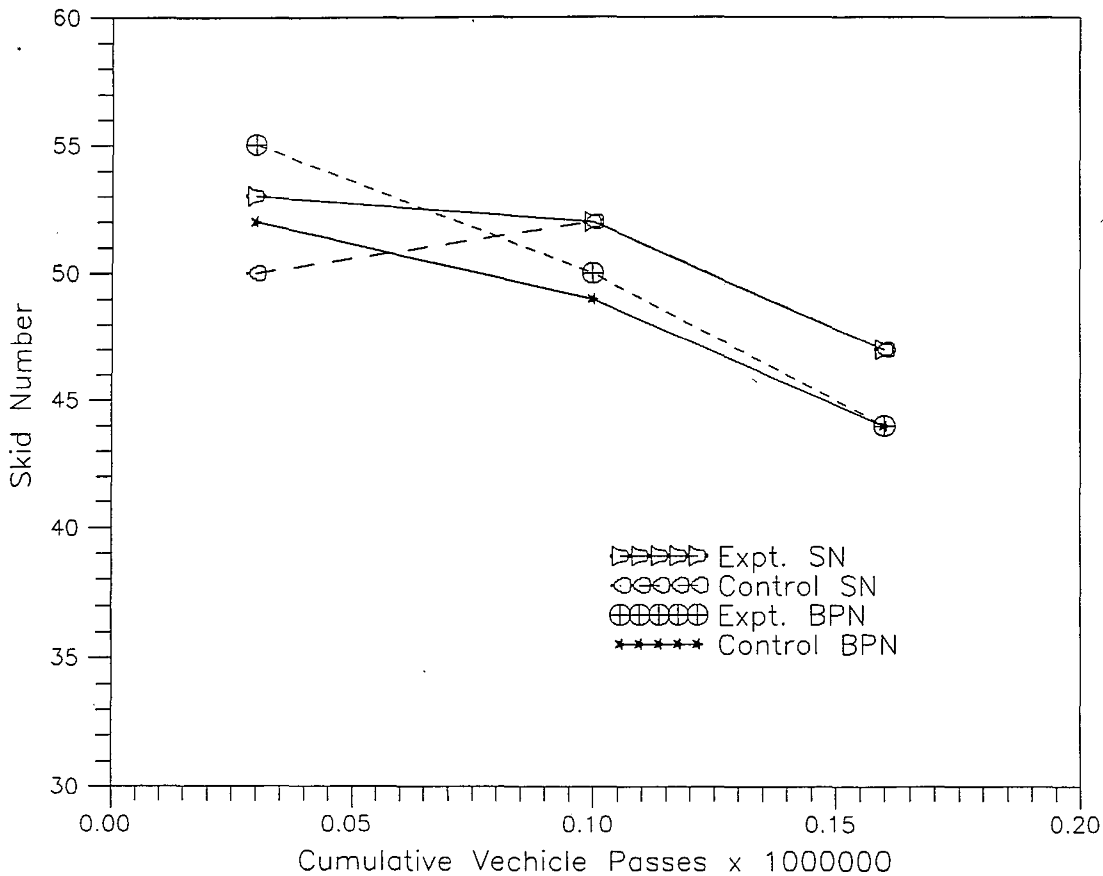

Figure 25. SN versus cumulative vehicle passes (Hale County).

INSOLUBLE RESIDUE TEST RESULTS

Three samples of each limestone type, weighing 500 grams each were tested. Six normality HCL solution was used to dissolve the samples in accordance with procedure outlined in ASTM D3042-86. Weights retained from the three samples were averaged to obtain the insoluble residues summarized in Table 21. The percent retained for all the limestone aggregates are small indicating very little noncarbonate material or impurities. This should mean that the aggregate will not provide good long term skid resistance (35). However, laboratory tests and field performance do not show that the limestone aggregates are particularly polish susceptible. Differences between the aggregate sources were also small indicating that differences in polish susceptibility should also have been small.

PETROGRAPHIC ANALYSIS

Petrographic analyses were conducted on the limestone and gravel coarse aggregates of all three counties by the Geology Department of Auburn University. The petrographic analysis shows that the Chilton and Hale County limestones are dolomitic in nature. The limestone aggregates of all three counties were soft with Mohs hardness in the range of 3.5 to 4. Silicious minerals were less than 1% and porosity was less than 1-2% for all carbonate aggregates. The Fayette County gravel consisted of 75% chert. Chilton and Hale County gravel were mainly quartz and quartzite with some amount of chert. Detailed petrographic reports are included in the Appendix.

Table 21

Acid insoluble residue test results

County	Original Wt. gms	Wt. Retained gms	% Acid Insoluble Residue
Fayette	500	1.33	0.27
Chilton	500	3.40	0.68
Hale	500	4.00	0.79

Table 22

Summary of effects of limestone.

County	Stability	TSR	Agg. BPN	Mix BPN	Pavt. BPN	Pavt. SN
Fayette	0	0	+	NR	0	0
Chilton	0	+	+	0	+	+
Hale	0	-	-	+	0	0

+ = Positive effect
- = Adverse effect
0 = Negligible or conflicting effect
NR = Not run

The petrographic analysis confirmed the findings of the acid insoluble residue test results and identified Hale and Chilton County limestones as dolomitic in nature. The analysis determined the mineral composition of each coarse aggregate and this finger printing is expected to be useful in future for reference purposes. Since the limestone aggregates were found to be soft (Mohs H= 3.5-4), the high microtexture levels observed on the limestone-gravel blend specimens during the polish testing could be attributed to differential wear.

VI. CONCLUSIONS AND RECOMMENDATIONS

CONCLUSIONS

In this study, tests were conducted to study the effects of limestone coarse aggregate (substituted partially for gravel coarse aggregate) on structural, moisture resistance and skid properties of asphalt concrete wearing mixes. On the basis of these tests Table 22 was prepared and, the following conclusions inferred :

1) The effects limestone coarse aggregate on mix stability was not well defined. This was partially due to inconsistences in mix composition and voids. The limestone mixes generally had increased stability as percent limestone increased, but increased stability was not demonstrated at lower limestone contents.
2) Resistance to moisture damage increased for the Chilton County mix, remained relatively unchanged for the Fayette County mix and decreased for the Hale County mix as percent limestone increased.
3) Laboratory polish tests on aggregate and mix indicated positive or no effects of limestone with the exception of the aggregate tests for the Hale County aggregates which indicated that limestone had a detrimental effect on polish resistance.

4) BPN and SN field tests on the pavement surfaces indicated that limestone did not adversely affect skid resistance. For Chilton County there were indications that limestone improved skid resistance.

RECOMMENDATIONS

It appears possible to use limited amounts of limestone coarse aggregate in surface course mixes without adversely affecting skid resistance properties. Based on this study, a maximum of 25% coarse aggregate is recommended.

Additional research is needed to expand and verify results from this study. Specifically, test sections should be designed, constructed and skid resistance monitored with limestone included as coarse and fine aggregate. The test sections should be designed to establish limits for amount and size of limestone aggregates. Polish characteristics of the limestone should be considered and sources with a range of BPN and acid insoluble residues should be included to begin establishing criteria for limestone quality.

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APPENDIX

PETROGRAPHIC ANALYSIS REPORT

Petrographic Descriptions of Carbonate Aggregates

Sample 1: Chilton County

DOLOSTONE/DOLOMITIC MARBLE: Light gray (N7) to medium light gray (N6); recrystallized, little original depositional texture preserved.

~95% of clasts are composed of <u>coarsely</u> <u>crystalline</u> (range 0.25-0.50 mm; average 0.35 mm), equant to subequant, angular rhombohedral to subrounded/rounded dolomite (H=3.5-4.0), (rounded crystals reflect the replacement of original fossil fragments and/or oolites); coarse silt- and/or sand-sized quartz and other siliceous minerals generally absent (<1% where detected); *porosity <1-2%.

~5% of clasts are composed of <u>medium</u> <u>crystalline</u> (~0.12 mm), angular rhombohedral to subrounded dolomite (~98%) and rare rounded to well-rounded, medium- to coarse-grained (0.25-0.60 mm) quartz (~2%); *porosity <1-2%.

*(dolomites are typically characterized by microscopic intercrystalline porosity related to volume reduction associated with replacement of calcite by dolomite; this type of pore space is not easily assessed by microscopy)

Sample 2: Hale County

DOLOSTONE/DOLOMITIC MARBLE: Light brownish gray (5YR 6/1) and dark gray (N3) to medium dark gray (N4); recrystallized, little original depositional fabric preserved.

~85% of clasts are composed of <u>finely</u> <u>to</u> <u>medium</u> <u>crystalline</u> (0.048-0.24 mm; average ~0.1 mm), equant to subequant, angular rhombohedral to subrounded/rounded dolomite (H=3.5-4.0) (rounded crystals reflect the replacement of original fossil fragments and/or other sand-sized carbonate grains); coarse silt- and/or sand-sized quartz and other siliceous minerals generally absent (<<1% if present); *porosity <1-2%.

~15% of clasts composed of <u>coarsely</u> <u>crystalline</u> (0.25-0.65 mm; average 0.5 mm), equant to subequant, angular rhombohedral to subrounded/rounded dolomite; coarse silt and/or sand-sized quartz and other silica minerals generally absent (<<1% if present); *porosity <1-2%.

Both lithologies characterized by healed fractures and solution seams; dark, fine-grained and/or opaque material (<2-3% of rock) occur along solution seams (clays, organics, sulfides, and/or other insoluble residues?) and in intracrystalline pore space (spent hydrocarbons?).

*(dolomites are typically characterized by microscopic intercrystalline porosity related to volume reduction associated with replacement of calcite by dolomite; this type of porosity is not easily assessed by microscopy)

Sample 3: Fayette County

LIMESTONE: very light gray (N8) to yellowish gray (5Y 8/1) and medium gray (N5).

~75-80% of clasts are composed of coarse, packed biomicrite (90-95% fossil fragments, 5-10% carbonate mud matrix); skeletal components include echinoderm (~60%), bryozoan (~25-30%), and other fossil (10-15%; brachiopod, foraminifera, coral, gastropod) fragments, all of which are calcitic; size of skeletal components ranges from 0.5 to 2 mm (average ~1.2 mm) (coarse calcarenite to fine calcirudite); shapes of skeletal components vary considerably from equant and rounded to irregular and elongate; Carbonate matrix (micrite) recrystallized to very finely to finely crystalline microspar; coarse silt- and/or sand-sized quartz and other silica minerals generally absent; fine-grained organic material and/or clays concentrated along stylolites/ solution seams (<1% of rock); porosity <1-2%.

~15-20% of clasts composed of coarse, poorly to moderately sorted biosparite (80-85% fossil fragments, 15-20% blocky spar cement); skeletal components include bryozoan (~50%), brachiopod (~25-30%), and other fossil (20-25%; echinoderm, foraminifera, and gastropod) fragments, all of which are calcitic; size of skeletal components ranges from 0.5 to 2.5 mm (average ~1.5 mm) (coarse calcarenite to fine calcirudite); shapes of skeletal components vary considerably from equant and rounded to irregular and elongate; medium crystalline (~0.2 mm) calcite cement; some micrite in intragranular pores; coarse silt- and/or sand-sized quartz and other silica minerals generally absent; porosity <1-2%.

<5% of clasts composed of sparse, biomicrite (90-95% micrite, 5-10% fossil fragments); micrite (carbonate mud) characterized by 40-50% finely crystalline (~0.03 mm) dolomitic rhombohedral microspar and 50-60% very fine grained carbonate mud; skeletal components include bryozoan and echinoderm fragments; coarse silt- and/or sand-sized quartz and other silica minerals generally absent; porosity <1-2%.

Petrologic Descriptions of Siliceous Aggregates

Sample 1- Fayette County

CHERT: grayish orange (10YR 7/4) to moderate yellowish brown (10YR 5/4).

~75% of clasts consist of dense, homogeneous chert; very finely crystalline (<0.016 mm) quartz; clast shapes range from subangular to subrounded; surfaces are smooth and exhibit conchoidal fracture.

~25% of clasts consist of porous chert; very finely crystalline (<0.016 mm) quartz; clast shapes range from subangular to subrounded; surfaces rough and pitted; porosity and surface textures reflect differential diagenesis of medium- to very coarse-grained (0.25-2.0 mm) skeletal components (mostly echinoderms) within the original limestone that was silicified.

Sample 2- Chilton County

QUARTZ AND QUARTZITE: very pale orange (10YR 8/2) to moderate yellowish brown (10YR 5/4) and light olive gray (5Y 6/1) to olive gray (5Y 4/1); and **CHERT:** grayish orange (10YR 7/4) to moderate yellowish brown (10YR 5/4) and moderate red (5R 5/4) to grayish red (5R 4/2).

~75% of clasts consist of coarsely to extremely coarsely crystalline (0.25 to >4.0 mm) quartz/quartzite; clast shapes range from subangular to rounded; surfaces smooth and frosted.

~25% of clasts consist of chert; very finely crystalline (<0.016 mm) quartz; clast shapes range from angular to subangular; surfaces mostly smooth with conchoidal fracture; some surfaces micropitted.

Sample 3- Hale County

QUARTZ AND QUARTZITE: grayish yellow (5Y 8/4) to grayish orange (10YR 7/4), rarely dusky yellowish brown (10YR 2/2) to olive gray (5Y 3/2); and **CHERT:** moderate yellowish brown (10YR 5/4) and grayish red (5R 4/2) to pale yellowish brown (10YR 6/2).

~65-70% of clasts consist of coarsely to extremely coarsely crystalline (0.25 to >4.0 mm) quartz/quartzite; clast shapes subrounded; surfaces vary from very smooth/polished to rough.

30-35% of clasts consist of dense, homogeneous to porous chert; very finely crystalline (<0.016 mm) quartz; clast shapes subangular to subrounded; surfaces vary from smooth with conchoidal fracture (~35%) to pitted (~65%); pitted textures reflect differential diagenesis of skeletal(?) components in original limestone that was silicified.