TRC 72

EVALUATION OF MINERAL FILLERS

AND ANTI-STRIP AGENTS

FINAL REPORT

TRANSPORTATION RESEARCH PROJECT NO. 72

by

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DECEMBER 1987

ACKNOWLEDGEMENTS

The project subcommittee expresses their thanks to the Bituminous Mix Design Laboratory for their effort in this study. The following persons were responsible for collecting the technical data in this report: Bill Wall, Terry Hardison, Reggie Cobb, C. A. Phillips, and Calvin Collins.

Special acknowledgement is given to Mr. Jim Brewster, Section Head of Highway Materials Design and member of the project subcommittee, who died November 19, 1985 after a long illness. Mr. Brewster had the unenviable task of educating all the engineers within the Materials and Research Division (including the author) about the art of designing asphalt mixes. His position may be filled, but he will never be replaced.

IMPLEMENTATION STATEMENT

The results of this study serve to provide valuable information regarding the effects of mineral fillers in changing the properties of the asphalt mixture. It also shows that other fillers such as hydrated lime, fly ash, and portland cement can be used as effectively or more effectively than limestone dust. The results also verify that mineral fillers can serve and perform the same function as an anti-strip agent. However, the anti-strip agent used in the evaluation did not compare favorably to the mixes using gravel and limestone aggregates. The reported "superior benefits" of using hydrated lime over other mineral filler was not realized with the tests conducted in normal Arkansas mixes. The most implementable product of this project is to realize that each mix is different and no particular additive should be pushed over another. Each mix should be evaluated by itself with no blanket single additive requirement. It is doubtful that one additive can be found to cure our stripping problems with every aggregate source.

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INTRODUCTION

Arkansas has a variety of aggregates that are used in asphalt mixes and bituminous surface course. Large limestone deposits exist in the northern one-third of the state, sandstone in the west central part of the state, and gravel in the southern and eastern sections. Novaculite is found in 3 counties of the state and syenite in the Little Rock area.

Arkansas has experienced some significant stripping problems with most of these aggregates. This has led the Arkansas State Highway and Transportation Department to require the addition of either an antistrip agent or mineral filler in standard asphalt mixes. Furthermore, mix designs for heavily traveled highways require both mineral fillers and antistrip agents. The adoption of these requirements have led many highway engineers to ask what affects these changes have upon mixes and how can these additives be used effectively.

The purpose of this report is to provide a basis of understanding about the effects of adding mineral fillers to mixes with the aggregates mentioned above. The use of mineral fillers not normally added to asphalt mixes at present are also studied. The report examines the effectiveness of various chemical antistrip agents by means of a boiling water test and compares the performance of limestone dust, hydrated lime, fly ash, and portland cement by evaluating the moisture susceptibility in asphalt mixes by the immersion compression test.

CHAPTER II

REVIEW OF LITERATURE

In order to evaluate the data contained it is necessary to have a thorough understanding of Marshall mix properties in addition to knowledge of mineral fillers and antistrip agents. A brief review of the Marshall mix properties and their effects on pavement performance is offered in this literature review.

Professor Thomas White of Purdue conducted a historic review of the Marshall method in his 1985 paper (1). He reported that the Marshall method was originally developed in the early 1940's by Bruce G. Marshall of the Mississippi Highway Department. The early Marshall procedure was said to have consisted of stability and flow measurements ofr design purposes. The compaction procedure was given as 25 blows with a standard Proctor hammer and an application of 5000 lb. static load for two minutes. Early flow measurements were found in 1/32" rather than the current 0.01 in. Bruce Marshall credited the extensive research conducted by the Waterways Experiment Station (WES) in Vicksburg, Mississippi with the development of the procedure to the currently used form (2). In this 1949 paper, Mr. Marshall described 5 factors that he termed essential to the development and design of a satisfactory mixture for paving. These factors were given as: 1) compaction of specimen, 2) per cent voids in the mineral aggregate, 3) density, 4) stability measurement, and 5) flow value. Marshall believed that no single factor could be used as a criterion for controlling the quality of a paving mixture and that all these factors should be addressed.

Marshall stated that the compactive effort applied in the production of a test speciman has a direct bearing on all the physical properties measured by the Marshall method. If the compaction effort applied to a specimen is large, the aggregates will be forced together very intimately. This was found to cause the total volume of void spaces between the aggregates to be relatively small in proportion to the total, solid aggregates. By comparison, lesser quantities of compactive energy will result in a higher aggregate-void content. Asphalt cement enters and fills these voids to a certain degree leaving a small volume of air voids. For this reason, greater compactive energy results in less asphalt cement being required to fill the voids in the aggregates. Furthermore, if the asphalt content of the mixture fills the aggregate voids to the maximum degree, at time of construction, the pavement will increase in plasticity as traffic further consolidates the mix. Beyond a certain point the increase in plasticity will result in shoving, displacement or rutting of the pavement under repeated traffic loads and stresses.

Mr. Marshall further stated that compactive effort applied to specimens for establishing the asphalt content and testing purposes must produce density equivalent to that which will be ultimately developed under traffic. The Marshall procedure for compaction has been correlated to reproduce ultimate compaction by traffic to the nearest practical degree. For this reason, compaction was not used as a variable factor and the physical properties are established at this predetermined compactive effort.

Marshall placed considerable emphasis on the percent voids in the mineral aggregate (VMA). He defined VMA as an expression of the volumetric proportion of voids in relation to the solid aggregates in a compacted state in the mixture. He further described the VMA as an opposite expression to the density of the combined aggregate mass. The total volume of asphalt cement and air voids in the compacted aggregate mass represents the VMA, and calculations of this qualitative value was given by the following formula:

VMA = 100 - SVD plus (d)(w)/(g) where:

VMA = percent voids in compressed mineral aggregates

SVD = percent of solid volume, or theoretical, density

d = bulk specific gravity of compacted specimen

W = percent by weight of asphalt cement in total mix

G = specific gravity of asphalt cement

The quantity of asphalt required for a given aggregate was explained as being intimately related to the VMA due to the fact that asphalt acts as a void filling material. It was believed that, from the standpoint of economy, the VMA should be reduced to the lowest practical degree. This reduction was said to result in a superior pavement structure as well as to reduce the quantity of asphalt required in the mixture. Marshall stated that no limits can be established for VMA for universal application because of the versatile application of bituminous materials to many types and gradations of aggregates.

Marshall explained in detail the change in VMA as asphalt cement is added. The lowest VMA is found theoretically when the compacted aggregate mass contains no asphalt. As asphalt is added to the mix, the surface of the aggregate particles become coated with asphalt films. These films separate the aggregate particles from their most compacted state, thereby increasing the VMA. This process continues and further prevents aggregate consolidation until the asphalt films become thick enough to act as a lubricant under the compactive effort. From this point, additional increments of asphalt further lubricate the aggregates and cause a reduction of the VMA until the voids are filled to their maximum practical degree with asphalt. After the voids are filled, further additions of asphalt cause a separation of aggregate particles causing the VMA to increase again with each increment of asphalt.

Mr. Marshall also reported the effect that mineral filler has upon VMA and attempted to rationalize its effect. Marshall found that VMA is reduced by progressive increases in the quantity of material passing the #200 sieve up to a point. Beyond this point, further increments of mineral filler would increase the VMA. This is reported as the result when the material passing the #200 sieve is increased beyond the quantity required to fill the voids in the sand fraction retained on the #200 sieve. The addition of an excess of mineral filler will produce an inherently plastic mix having physical properties which will be highly critical to slight variations in asphalt content.

Mineral filler consisting of coarse particles, even though the particles are finer than .074mm, would not be as effective in all cases as the more finely pulverized mineral fillers. This is because the coarser mineral filler particles are larger than the normal vild spaces between the aggregates coarser than the #200 sieve, thereby preventing their maximum consolidation.

Marshall believed that the quantity of asphalt required in a mixture should be determined by a plotted curve of the wieght per cubic foot of total mixture at various asphalt contents. The apex of this curve is used to locate the required asphalt content. This plot of density versus asphalt content should reflect the fact that as the asphalt content is progressively increased it repleaces air spaces until the aggregate voids are filled to the maximum. Further increases in the asphalt beyond this point simply prevents maximum consolidation. Since asphalt is lighter than the aggregate, the density of the mixture is decreased by the addition of asphalt in excess of that required to fill the aggregate voids.

Marshall reported that the stability value expresses the structural strength of a compressed paving mixture. He believed that by establishing a high minimum stability for a given locality, this physical property will prevent the use of inferior types of aggregates or those containing excessive VMA. Excessive VMA was also reported to give high flow values before reaching optimum asphalt content. Since the flow value is an index to plasticity, this type of gradation was reported unsatisfactory.

Density alone should not be construed as a criterion of quality of the paving mixture according to Marshall. He demonstrated that density can be obtained in a mix offering little resistance to shear forces. The proper method of attaining high density was reportedly by reducing the VMA by improving the overall aggregate gradation so that a relatively low asphalt content will be required. Marshall believed that under this condition the mixture will be workable and will possess high resistance to distortion or shoving and the highest possible shear resistance.

Marshall stated that his stability value gives the structural strength of a compressed paving mixture and is an index to aggregate quality. It is primarily affected by the asphalt content and the gradation and character of aggregates in the mix. Marshall believed that by establishing a high minimum stability for a given locality the use of inferior types of aggregates or those containing excessive VMA will be prevented.

Mineral Filler

In 1962, Dr. David G. Tunnicliff published a paper reviewing the description and use of mineral filler (3). Tunnicliff found reports on the use of mineral fillers dating back to the late 1870's. In the early 1900's a definition of mineral filler was offered. This definition was that a good filler should contain at least 60 percent of its weight of actual dust ($<$.05mm), and preferably over 70 percent. In 1913, Tunnicliff found that filler was redefined as part of the mineral aggregate with at least 75 percent passing the number 200 sieve and at least 66 percent remaining suspended in water for 15 seconds. The prevailing thoughts on the usefullness of mineral filler in the 1920's was summarized as follows:

> "... Most authorities regard mineral filler as a constituent whose function it is to extend the grading of the aggregate down to almost inconceivably fine sizes. Regarded in this light, it becomes purely a space filler, occupying the voids between the larger particles and contributing to the density of the mixture. Present specifications are based on this theory and generally require the filler material to meet a requirement of fineness. Certain indications, however, have been developed, which indicate that gradation, surface texture, and the shape of the mineral filler particles may effect the compressibility and water proofness as well as the strength of mixtures, but this angle still remains to be fully substantiated."

By the 1930's Tunnicliff reported that the conception of filler had changed to accept that filler forms a colloidial suspension in the bitumen, and in this way becomes a part of the bitumen itself. A list of acceptable filler materials was given. These fillers included limestone dust, portland cement, slate dust, silica flour, brick dust, granite dust, flue dust, slag, anhydrite, fuller's earth, coal dust, or anything fine enough to be chemically inert.

Tunnicliff concluded that a satisfactory definition of mineral filler was not found; however, he proposed the following definition:

> "Filler is mineral material which is suspended in asphalt cement resulting in a cement of stiffer consistency."

Tunnicliff believed that correct methods for evaluating and proportioning filler for paving purposes must also be based on the change in consistency, or stiffening in the binder.

In 1969, Mr. V. P. Puzinauskas of the Asphalt Institute reported the results of his investigation into the effects of mineral filler in asphalt paving mixtures (4). Mr. Puzinauskas believed that mineral fillers play a dual role in paving mixtures. They act as mineral aggregate and fill the voids between larger aggregate particles to strengthen the mix. Also, filler particles smaller than the thickness of asphalt films combine with the asphalt to form a high consistency binder. The water sensitivity of paving mixtures containing different types and different concentrations was found to vary over a wide range. He recommended Immersion-Compression testing to supplement the mix design.

The Texas State Department of Highways and Public Transportation (TSDHPT) initiated a study to investigate the characteristics of mineral fillers (5). The TSDHPT defined mineral filler as follows:

> "Mineral filler shall consist of thoroughly dried stone dust, slate dust, portland cement, fly ash or other mineral dust approved by the Engineer. The mineral filler shall be free from foreign matter. Fines collected by baghouse or other air cleaning or dust collecting equipment may be permitted as mineral filler in the asphaltic mixture up to 2 percent, provided that the passing No. 200 master gradation limit is not exceeded. When these fines are permitted in the asphaltic mixture, they shall be introduced in the same manner prescribed for other mineral fillers."

The grading requirements of mineral fillers is given by the TSDHPT as follows:

	Percent by Weight or Volume
"Passing No 30 sieve	95 to 100
Passing No 80 sieve, not less than	75
Passing No 200 sieve, not less than	55"

During the course of this study, 7 different types of mineral fillers were tested. The mineral fillers tested included portland cement, fly ash, limestone dust, and hydrated lime. It was found that some limestone dusts were too coarse to meet the above gradation. Also, reported was the importance of voids in mineral fillers. This parameter was believed to be of much importance when evaluating the influence of filler type on the asphalt paving mixture. It was discovered that cement allowed the most voids and fly ash the least. Fly ash was thus viewed as a mineral filler providing the least strengthening.

Mr. W. Huekelom of Shell Research in Amsterdam reported on the effect of filler in asphalt mixes (6). He stated that the addition of filler results in a decrease in percentage of voids in standard Marshall compacted specimens. The type of filler also governs the optimum bitumen content. He further stated that the use of a standard compaction effort also introduces an effect on the workability of the mixes which depends on the proportion and the type of filler added. The results of this study led Hueklom to conclude that the volume of free bitumen left after filling the voids in the filler is a more direct measure of mix properties than the total volume of bitumen. Furthermore, the volume of asphalt present in the filler voids can be regarded as filler and should be subtracted from the VMA to relate with mix properties.

The Arkansas State Highway and Transportation Department (AHTD) requirement for mineral filler in the surface course is stated as follows (7):

> "At Least one-half of the fraction passing the No. 200 sieve shall comply with the requirements for mineral filler and in no case shall the mineral aggregate contain less than 5 percent mineral filler."

In 1982 , a provision was added to allow a heat stable antistrip in lieu of added mineral filler. Mineral filler was required on binder and surface courses along with anti-strip agents when designs called for mixes to withstand heavy traffic. The required addition of mineral filler was given as between 2 to 4 percent. In addition, mineral fillers are to meet the requirements of AASHTO M17. AASHTO M17 states:

> "Mineral filler shall consist of finely divided mineral matter such as rock dust, slag dust, hydrated lime, hydraulic cement, fly ash, loess, or other suitable mineral matter. At the time of use it shall be sufficiently dry to flow freely and essentially free from agglomerations.

Mineral filler shall be graded with the following limits:

Sieve	Percent Passing (by weight)
30	100
50	95 to 100
200	70 + 100 "

No other mineral filler requirements are given by the AHTD or AASHTO.

Moisture Damage

There are three different distress mechanisms that have been observed to result from the detrimental effects of moisture on asphalt (8). These three mechanisms are stripping, ravelling, and shelling. Stripping may be defined as the deboning of asphalt from aggregate due to water. There are five different types of stripping that have been defined (9). Of these five types of stripping, detachment and displacement may be the two types of stripping that allow adequate prediction by laboratory testing of compacted Marshall specimens.

Taylor and Khosla conducted a comprehensive literature review of moisture damage to asphalt pavements in 1983 (10). The report includes a brief discussion of stripping mechanisms, use of anti-strip agents, and tests to predict moisture susceptibility. They reported that two stages of failure from stripping can occur in asphalt pavements. The first atage is stripping failure, and the second stage is failure of the pavement under traffic. Many asphalt pavements experience stripping failure within the mix without structural failure of the pavement. If stripping becomes excessive, loss of strength may result in excessive deformations caused by repeated loading. This can lead to complete disintegration of the roadway. Evidence suggests that a stripped pavement will not fail unless the pavement structure has pronounced flexibility. Also, numerous investigators were stated to have observed that is a stripped asphalt pavement is exposed to dry environment, a certain amount of healing will take place.

The researchers reported that dozens of tests to predict the moisture susceptibility of asphalt mixtures had been developed. They noted that none of the tests developed to date have received wide acceptance, and surmised that this is due to the low reliability of test methods. The tests were divided into three broad categories. These categories are immersion tests, coating evaluation tests, and immersion-mechanical tests.

The qualitative coating evaluation tests involve the immersion in water of loose coated mixtures, typically having a specified aggregate gradation, with or without agitation of the immersed mix. In each of these tests, the asphalt mixture remains immersed for a specified period of time, and at the end of that time a visual estimation is made of the percent coating retained on the aggregate. The main advantage of qualitative coating evaluation tests is that they are simple to perform, require little equipment, and can be performed in a short period of time. However, this type of test has been criticized for its lack of correlation with field performance.

The immersion mechanical test measures changes in a specified mechanical property of compacted mixtures, such as shear strength, tensile strength, flexural strength, compressive strength, etc., caused by exposure to moisture. The moisture conditioning may be applied by submerging samples in water for a prescribed amount of time, or by applying a vacuum before introducing water. The compressive strengths or tensile strength of samples conditioned versus unconditioned is used to calculate an index of retained strength. An acceptable range of the index of retained strength has been reported from 65 to 75 percent. The main benefit of immerison-mechanical tests is that they allow the use of a mixture which is representative of the mix which will be utilized in the field, and which can be compacted to a density comparable to the proposed field density. One reported restriction of immersion-mechanical tests is that identical specimens cannot be molded. Furthermore, no quantitative correlations between the results of immersion-mechanical tests and field performance of bituminous pavements have been developed.

In 1985, a workshop on the moisture damage of asphalt concrete mixtures was held in Cincinnati, Ohio. Present at the meeting were some of the most knowledgeable highway engineers in the field of asphalt mix design and performance. The most notable finding from this meeting was the attendee's lack of an agreement on any issue concerning moisture damage (11). A universal definition of the term "moisture damage" could not even be agreed to. There was a long discussion of existing test methods used to predict moisture damage and the relation of these laboratory procedures to actual field performance. In general, there were good and bad points discussed of every type of stripping test on the agenda.

The question of using a percentage of retained strength to judge the acceptability of the test results was explored. An example was given of Mix A which had a dry strength 400 psi and a wet strength of 300 psi, for a retained immersion-compression strength ratio (wet/dry) of 0.75. Mix B had a dry strength of 600 psi, a wet strength of 400 psi, and a strength ratio of 400/600 = 0.67. A discussion ensued as to which was the better mix, A or B. If a retained strength of 0.70 is required by the specification, Mix A is acceptable while Mix B is not. There was no strong agreement whether to use dry strength values, wet strength numbers, retained strength ratio, or a combination of these values when assessing the validity of the test results. However, most individuals felt that the strength ratio values should not be used alone or without regard to the actual wet strength numbers.

The lack of agreement between the engineers at this workshop indicated that there is still much work yet to be done in the area of moisture damage to asphalt pavement. The attendees showed that no one test method has received wide spread acceptance. Furthermore, it seems unlikely that one test will receive this acceptance in the near future.

Dr. Miller Ford of the University of Arkansas reported on his work with immersion-compression testing of Arkansas mixes using 5 different sources of aggregate (12). Ford found that the most significant factor affecting the Index of Retained Strength was the percent air voids in the mix. The following conclusion was drawn from his research effort.

> "The air void content of the compacted asphalt mixture greatly influences the result of the immersion-compression test. Specimens with greater than 5 percent air voids would have a retained strength of about 75 percent."

The Montana Department of Highways reported on an investigation into predicting moisture damage to asphalt mixes in 1978 (13). In this study, a variety of specimen sets of different aggregates, asphalts, fillers, and additives were molded for testing. These samples were tested using the following; 1) E Modulus, 2) Immersion Compression, 3) Marshall Method, 4) Maximum Tensile Stress, and 5) Resilient Modulus.

The results of the study indicated that Resilient Modulus and Tensile Stress testing were the better methods of evaluating the susceptibility of asphalt mixes to moisture damage. It was stated that the use of Immersion Compression ratio as a test for moisture susceptibility is not fully reliable. However, the Immersion Compression test produces enough useful data for detecting moisture susceptibility that its use should be retained. Immersion-Compression ratios that were low did correspond to susceptible mixes. The more moisture resistant manufactured mixes were usually found to yield the highest Immersion Compression wet strength. An extension of the Immersion Compression test is needed to require that mixes with the highest wet strength be used.

During the laboratory testing phase, it was found that the variation in the amount of material passing the 200 seive (-200M) affected the moisture susceptibility of the mix. It was believed that 0% - 200M was undesirable, 1-2% acceptable, 3-4% undesirable, and 4-11% satisfactory. The higher percentages of -200M greatly reduced moisture susceptibility if effectively compacted.

Hydrated Lime

Hydrated Lime has been used for mineral filler in asphalt concrete for at least 70 years (3, 14). During the past thirty-five years hydrated lime has been used in asphalt mixes as an anti-stripping additive. It is believed that hydrated lime not only functions as mineral filler and anti-strip agent, but also as a neutralizing agent. All three functions of lime can prevent stripping.

There are several types of lime that are used in asphalt concrete. The most common is high calcium hydrated lime, $Ca(OH)$. Two types of dolomitic hydrated lime are also used, $Ca(OH)_2MgO$ and $Ca(OH)_2Mg(OH_2)_2$. Quicklime CaO, can also be used to produce $Ca(OH)$ in the field. Hydrated lime is a powdered substance that meets the requirements of mineral filler and can reduce the stripping potential of mixes by densifying the mix to prevent the intrusion of water. Chemically, hydrated lime is a strongly alkaline substance that reacts with aggregate surfaces to promote bonding of the aggregate to the surface (15). It has been reported that the neutralizing effect can reduce the rate of aging in pavements (14).

Lime is normally transported in semi-trailer tank trucks equipeed with pneumatic transfer systems for unloading. The lime is usually stored in a silo that may be equipped with penumatic vibrating pads to help the lime flow. Other silos may use air agitation systems. Lime silos or tanks must be equipped to control particulate emissions while being filled. Perhaps the greatest problem associated with lime storage is the dampening of the lime (14). Damp line will not flow and must be cleaned from the silo before it can be used again.

Hydrated lime is usually applied directly on the aggregate in slurry form. Lime may be added to the asphalt or added to the aggregate in a dry form. However, these results are not as dramatic as those from slurry applications. The normal dosage of lime ranges from 1 to 2 percent by weight of aggregate.

The Georgia Department of Transportation (GDOT) began requiring the use of hydrated lime in some asphalt mixes in 1986 (16). Initially, lime was added to the mix along with liquid antistrip agents. However, after further investigation, the use of liquid additives with lime was discontinued. Some behavioral difference in lime treated mixes were reported. The workability of the lime mixes had changed requiring a rise in laydown temperature. Hand work was reported to be more difficult. Also, foaming of the roadway after a rain has been noticed in some instances.

It was reported that the GDOT used approximately 50,000 tones of lime in asphalt mixes in 1983. Furthermore, 80 asphalt plants have installed lime injection units. Fifty-six of these are batch plants. Lime was generally added to Georgia mixes at a dosage of near 1 percent.

Antistrip Agents

Antistrip additives are routinely used by many state highway agencies to improve the water resistance of asphalt concrete mixtures. There are approximately 27 antistripping additive manufactureres with 116 approved products used by agencies across the country (17). Most of these additives are liquids designed to promote adhesion between asphalt and aggregate.

All chemical antistrip agents are amines or compounds containing amines and are strongly basic compounds derived from ammonia. Many are described by asphalt specialists as cationic surfactants that enhance adhesion after migration to the aggregate surface (8, 18). After migration, the surfactants displace moisture and make the aggregate prefer asphalt rather than water. It appears that the concentration of the antistrip agent in the asphalt can greatly affect its performance. If the concentration of chemical dissolved in the asphalt is in excess of that needed to satisfy all of the absorption sites of the aggregate, a reorientation can occur creating a mechanically weak, water susceptible shearplane. The concentration of agent needed to promote bonding may also be affected by the properties of the asphalt. There is also the problem of asphalt-additive compatibility, which is beyond the authors ability to adequately describe (19). It is believed that a great majority of the antistripping agent is never able to migrate to the aggregate surface while the viscosity of the asphalt is low. Consequently, the additive cannot migrate to the surface of the aggregate after the mix has cooled because its viscosity has increased dramatically. In many instances, the time given for additive migration is less than three hours. It is estimated that only 30 to 40 percent of the original concentration of antistrip agent is performing in the proper manner.

Generally, chemical additives are added to the asphalt cement at a rate of approximately 0.5 to 1.0 percent by weight of asphalt. While the widespread usage of antistrip agents suggests that antistrip agents are effective, there are several factors affecting their performance. It is generally known that the resistance to stripping may be drastically changed if either the asphalt cement, aggregate, or additive is changed. It has also been found that aggregates may absorb compounds as they age that may increase or decrease their stripping potential after crushing (17). This in turn affects the needed dosage of antistrip agent.

Heat stability at usual working temperatures is said to be characteristic of all antistripping additives. To be heat stable, the additive must not contain compounds which react with some component of the asphalt (8). All manufactures have been reported as claiming heat stability of their products.

The Louisiana Department of Highways has performed research into the use of antistripping additives in lieu of mineral fillers in asphalt mixes (20). The scope of this study was confined to sand-gravel wearing course mixes from six major sources generally used in Louisiana. The various mixes were evaluated by use of the following criteria:

1. Marshall Stability and Flow
2. Percent Voids in the Total Mix
3. Index of Retained Strength
4. Visual Stripping

The results of this study led the Louisiana researchers to conclude that the use of an antistripping additive and no mineral filler may result in loss of stability and density. However, retained strength values did not indicate any detrimental effect of water on mixes without filler. It was recommended that replacement of mineral fillers with antistrip agents should not be implemented using these test results.

Chapter III

TEST METHODS AND MATERIALS USED

The test methods employed and materials used were consistent with Arkansas State Highway and Transportation Department (AHTD) methods and specifications. Standard test procedures used in this study include:

1. Marshall Method of Mix Design as given by The Asphalt Institute's publication MS-2 and meeting the requirements of AASHTO T245.
2. Water Sensitivity Test for Compacted Bituminous Mixtures - AHTD Test Method 132
3. Test for Effectiveness of Heat Stable Antistrip Additive (Georgia Boil Test)
4. Sieve Analysis of Fine and Coarse Aggregates - AASHTO-T27 and AASHTO T-88

These test methods are shown in Appendix A.

Materials

Five different aggregate types representative of the aggregates typically used in Arkansas were selected for this study. The five aggregate types were limestone, sandstone, novaculite, syenite, and gravel. TOSCO AC 30 asphalt cement was used for the evaluation of mix properties and the effects of anti-strip agents upon the viscosity of the asphalt. MacMillan AC 40 was used in the viscosity evaluation only. Both asphalt cements are produced in Arkansas and predominately use crude oil from southern Arkansas in the manufacture of asphalt. Several antistrip agents were evaluated for stripping resistance. These agents included Indulin 772, Indulin 773, Kling Beta XP-251, PermaTac Plus, Unistrip 85, Kling Beta W, and Unistrip 120. In addition, Kling Beta W and Unistrip 120 were used to evaluate viscosity changes when added to the asphalt cement. Permatac Plus was used in the mix design portion of the evaluation.

Four different mineral fillers were used in this study. These minerals were hydrated lime, limestone dust fly ash, and portland cement. The limestone dust and hydrated lime came from the Batesville Lime Company, the fly ash from Chem Ash Products, and the portland cement from Foreman Cement. All materials are produced and marketed in this state.

Tests

The Marshall mix testing was extended to encompass more data points because of the effects of different mineral fillers on the optimum asphalt content. For example, 5 samples each were molded at asphalt contents of 3.5, 4.0, 4.5, 5.0, 5.5, and 6.0 percent for the limestone aggregates. Since 7 different limestone mixes were molded, approximately 250 samples were molded. Overall, over 1100 Marshall samples were molded and tested for this study. The design curves showing AC content versus stability, retained stability, flow, density, VMA, and air voids for all aggregates and additive combinations are given in Appendix B.

The gradation of the aggregates was held constant in this study. However, the addition of mineral filler did change the amount of -200 material in the mix. The amount of material passing the 200 sieve was figured to be 6%. The addition of 5% limestone dust increased this percentage to 7.7% while the addition of 5% portland cement increased this amount to 8.5%. The gradations used with each additive is shown in Table 1.

The application of Stoke's Law in determining the particle size distribution of mineral fillers as described in AASHTO T-88 was very difficult. The fillers could not be dispersed and soaked for 12 hours as dexcribed in AASHTO T-88. The mineral fillers were found to gel in that 12 hour period. Moreover, the fly ash and cement hardened in that period of time to look like fresh concrete. In an effort to overcome this problem the amount of stock solution containing sodium hexametaphosphate was double and dispersed for 90 seconds instead of 60 without a soaking period. The results of the particle size analysis are shown in Appendix B.

Table 1.

Combined Aggregate Gradations
with each additive

Additive	Percent retained on sieve							
	3/4	1/2	3/8	4	10	40	80	200
none	0	10	18	37	55	74	87	94
0.5% antistrip	0	10	18	37	55	74	87	94
5% Limestone Dust	0	10	18	37	55	74	87	92.3
5% Fly Ash	0	10	18	37	55	74	87	91.6
5% Portland Cement	0	10	18	37	55	74	87	91.5
2% Hydrated lime	0	10	18	37	55	74	87	92.9
3% Hydrated lime	0	10	18	37	55	74	87	92.4

The mineral fillers and antistrip agents were added to the mix in the same manner as normal mixes designed for the Arkansas State Highway and Transportation Department. The liquid antistrip agent was added to the asphalt and stirred while being heated over a bunsen burner. The asphalt antistrip was added to the aggregate immediately before mixing. Mixing and compaction temperatures were maintained in Accordance with AASHTO Test Method T245. The mineral fillers, including the hydrated lime were added in dry form to the aggregate before the asphalt was introduced. Very little dry mixing was performed. No attempt was made to "slake" the hydrated lime onto the aggregate.

The molded samples of each individual data point were divided into two sets at random. One set was tested for bulk density, flow, and stability. The other set was vacuum saturated and immersed in 140^{o}F water for 24 hours before testing. These samples were then tested for retained stability and flow.

The Georgia boil test used to determine additive effectiveness was performed on 13 different antistrip agents and 20 different asphalt-antistrip combinations. Only one aggregate source, syenite, was used. The gradation of the aggregate was held constant. This gradation is given in Appendix A. Four different asphalts were used. Tosco AC30 and McMillan AC40 were used in most of the evaluation. Delta AC20 and Exxon AC30 were each used once in the evaluation. The change in penetration, absolute viscosity, and kinematic viscosity of four asphalt-antistrip combinations were measured when the boil tests were performed. Therefore changes in penetration and viscosity initially and at 24 hour and 96 hour aging were measured.

Chapter IV

DISCUSSION OF TEST RESULTS

The combined aggregate gradation was held constant throughout the mix designs for the different aggregate sources. This was done to reduce the already large amount of variables under study. The mineral fillers were added to the aggregates at set percentages for the same reason. Hydrated lime was added at 2% and 3% of the mix. Portland cement, fly ash, and limestone dust were all added at 5% of the weight of mix. Likewise, the antistrip was added at 0.5% of the weight of asphalt in the mix.

These mixes were tested for normal Marshall mix design properties and for water sensitivity by immersion compression. Also, several antistrip agents were tested for effectiveness by use of the Georgia Boil Test. A brief look at the effect of two antistrip agents, unistrip 120 and Kling Beta W, upon the viscosity and penetration of the asphalt cement was also performed.

The Georgia Boil Test is a test of additive heat stability and aggregate coating for different antistrip dosage rates. For this study, syenite aggregate was used in the testing. Three different aging times of the asphalt-antistrip material was used in this boil test. The coated percentage of aggregate after a 10 minute boil initially, and after 24 and 96 hours of heating in a closed container. Two antistrip concentrations of 0.25% and 0.50% by weight of asphalt were used. The results of the Georgia Boil Test are shown in Table 2.

Several interesting observations can be made from the boil test results. Increasing the concentration of antistrip from 0.25 to 0.50% generally improved the test results but not enough to dramatically change the results. For example, if a 70% coating was required after 24 hour aging, in only two of

Table II

Georgia Boil Test Results

Antistrip Agent	Asphalt	Antistrip %	Percent Coating Initially	Percent Coating 24 Hrs.	Percent Coating 96 Hrs.
Indulin DP Special	Delta AC 20	.25	85	75	40
		.50	90	75	40
Kling Beta W-M	Exxon AC 30	.25	85	75	70
		.50	85	85	75
Kling Beta W-M	McMillan AC 40	.25	85	80	70
		.50	90	85	75
Indulin 772	McMillan AC 40	.25	85	25	20
		.50	90	40	20
Indulin 772	Tosco AC 30	.25	100	85	80
		.50	100	95	80
Indulin 773	McMillan AC 40	.25	45	30	20
		.50	50	20	20
Indulin 773	Tosco AC 30	.25	100	90	80
		.50	100	85	80
Nalco IRM-101	Tosco AC 30	.25	80	65	55
		.50	85	70	45
Nalco IRM-102	Tosco AC 30	.25	70	45	50
		.50	70	60	50
Kling Beta 500	Tosco AC 30	.25	95	75	50
		.50	99	85	65
Kling Beta 510	Tosco AC 30	.25	97	70	50
		.50	100	75	50
Kling Beta XP-251M	Tosco AC 30	.25	98	75	20
		.50	100	85	35
Kling Beta XP-251M	McMillan AC 40	.25	70	35	15
		.50	85	45	20
Permatac Plus	Tosco AC 30	.25	90	85	65
		.5	90	90	75
Unistrip 85	Tosco AC 30	.25	90	85	50
		.50	90	85	55
Unistrip 85	McMillan AC 40	.25	55	-	-
		.50	60	-	-
Kling Beta W	McMillan AC 40	.25	50	-	-
		.50	50	-	-
Kling Beta W	Tosco AC 30	.25	80	70	50
		.50	80	80	50
Unistrip 120	Tosco AC 30	.25	50	-	-
		.50	80	80	-
Unistrip 120	McMillan AC 40	.25	35	30	30
		.50	40	35	30

the seven failing cases did an increase in antistrip concentration help. The reason for this is that the increase in coating from the higher antistrip concentration averaged only 5%. The only significant gain was approximately a 35% coating increase from the Unistrip 120 antistrip with TOSCO AC30 asphalt.

The results of the boil test also show that asphalt-antistrip compatibility will also affect the results. In all six cases where the same antistrip was used and the asphalt was changed from TOSCO AC30 to McMillan AC40 the results varied greatly. The antistrips combined with TOSCO AC30 had far superior results in the boil test than the antistrips combined with McMillan AC40. The results of these are separated and shown in Table III. Furthermore, only one antistrip tested was effective with McMillan AC40. That antistrip agent was Kling Beta W-modified. Unfortunately, this antistrip agent was not mixed with TOSCO AC30 so no comparison could be made.

The change in penetration and viscosity of an asphalt with oven aging due to the addition of antistrip agents was tested using TOSCO AC30 and McMillan AC40 with Kling Beta W and Unistrip 120 antistrip agents. The results of these tests are found in Table IV. Upon examination of Table IV, it can be seen that, in general, antistrip agents tend to soften the asphalts. The penetration increases and viscosity decreases when antistrip agents are added. Also, as the amount of antistrip agent is increased, the asphalt is softened further. As expected, aging the asphalt did increase the viscosity and decrease the penetration. There was little difference in aging rates with different asphalt-antistrip combinations and concentrations. However, it is interesting to note that the Unistrip 120 did not initially affect the viscosity and penetration of the TOSCO AC30 asphalt as much as the Kling Beta W. Conversely, the Kling Beta W was found not to affect the McMillan AC40 as much as the Unistrip 120. The reason for this changing effect is not known.

Table III

Boil Test Comparison of Antistrip Effectiveness
Using McMillan AC 40 and Tosco AC 30

Antistrip Agent	Antistrip %	Asphalt	Percent Coating Initially	Percent Coating 24 Hrs.	Percent Coating 96 Hrs.	Change from McMillan to Tosco Initially	Change from McMillan to Tosco 24 Hrs.	Change from McMillan to Tosco 96 Hrs.
Indulin 772	.25	Mac*	85	25	20	15	60	60
		Tosco**	100	85	80			
	.50	Mac	90	40	20	10	55	60
		Tosco	100	95	80			
Indulin 773	.25	Mac	45	30	20	55	60	60
		Tosco	100	90	80			
	.50	Mac	50	20	20	50	65	60
		Tosco	100	85	80			
Kling Beta XP-251M	.25	Mac	70	35	15	28	40	5
		Tosco	98	75	20			
	.50	Mac	85	45	20	15	40	15
		Tosco	100	85	35			
Unistrip 85	.25	Mac	55	-	-	35	-	-
		Tosco	90	85	50			
	.50	Mac	60	-	-	30	-	-
		Tosco	90	85	55			
Kling Beta W	.25	Mac	50	-	-	30	-	-
		Tosco	80	70	50			
	.50	Mac	50	-	-	30		
		Tosco	80	80	50			
Unistrip 120	.25	Mac	35	30	30	15	-	-
		Tosco	50	-	-			
	.50	Mac	40	35	30	40	45	-
		Tosco	80	80	-			

*McMillan AC 40
**Tosco AC 30

Table IV

Change in Penetration and Viscosity
Due to Addition of Antistrip

Asphalt (type)	Antistrip (type)	Antistrip (%)	Penetration (cm) Init.	24 hr.	96 hr.	Absolute Viscosity (p) Init.	24 hr.	96 hr.	Knematic Viscosity (cs) Init.	24 hr.	96 hr.
Tosco AC 30	none	0	55	51	44	2569	2848	3790	500	514	675
McMillan AC 40	none	0	63	59	52	4303	4994	7337	679	696	845
Tosco AC 30	Kling Beta W	.25	64	52	44	2351	2751	3994	512	486	603
		.50	64	57	44	2260	2440	3643	439	510	563
		.75	68	60	46	2101	2400	3419	466	533	538
		1.00	70	65	47	1904	2289	3274	433	497	597
Tosco AC 30	Unistrip 120	.25	57	55	42	2508	2717	4181	470	522	588
		.50	59	59	44	2256	2850	3667	440	461	542
		.75	65	62	49	2138	2329	3444	462	487	544
		1.00	67	63	49	2011	2135	3155	420	444	531
McMillan AC 40	Kling Beta W	.25	60	58	52	4495	4514	6842	667	680	859
		.50	64	64	55	4022	4280	6017	666	665	1002
		.75	62	63	56	3887	4224	5891	675	671	919
		1.00	66	65	58	3540	3850	5535	735	673	938
McMillan AC 40	Unistrip 120	.25	66	68	49	4330	4675	8118	793	697	877
		.50	67	62	52	3936	4415	7535	639	672	829
		.75	70	64	50	3709	4114	7230	638	778	881
		1.00	72	69	52	3540	3788	6160	628	651	774

The Marshall mix properties of all the additive-aggregate combinations are found in Appendix B. Table V shows the effect of the fillers and antistrip agent upon the optimum asphalt content. The optimum asphalt content was figured by averaging the asphalt content of maximum density, maximum stability, and the 3 to 5 per cent void range. Upon examination of Figures 1 thru 5 and Table V, it can be seen that the addition of fillers can significantly alter the optimum asphalt content (AC). Even the addition of 0.5% Permatac Plus in the asphalt affected the optimum asphalt content. Fly ash was found to affect the optimum AC the most of all the fillers tested. The average effect of the 5% fly ash was to reduce the optimum AC by 0.77%. Likewise, all the fillers tended to decrease the optimum asphalt content while the antistrip agent tended to increase the optimum asphalt content.

In order to understand the effects of the filler upon the optimum AC, the individual percentages used to calculate the optimum must be studied. In the literature review it was reported that Shell Research (6) had found that the addition of filler results in a decrease in percentage of voids in Standard Marshall compacted specimens. This can be seen by looking at the asphalt content at 4% voids in Table V. The asphalt content found to produce 4% air voids for each mix is generally lower with mineral fillers than those mixes containing no mineral fillers. For example, the asphalt content needed for 4% air voids in mixes containing 5% fly ash averaged 0.9% lower than the mixes containing no additives. Likewise, portland cement, limestone dust, 2% hydrated lime, and 3% hydrated lime, lowered the asphalt content by 0.62%, 0.60%, 0.30%, and 0.22% respectively. Clearly, the change in optimum asphalt content is mainly due to the asphalt content is mainly due to the asphalt content required to produce 4% air voids. Table VI shows the densification of the mixes by the addition of the mineral fillers. It can

TABLE V

EFFECTS OF ADDITIVES UPON THE OPTIMUM ASPHALT CONTENT

AGGREGATE	ADDITIVE	AC MAXIMUM DENSITY	CONTENT FOUR % VOIDS	AT MAXIMUM STAB	I I I	OPT. AC CONTNT
SANDSTONE						
	NONE	6.0	4.7	5.0		5.2
	AS:0.5%	6.0	4.9	5.3		5.4
	HL:2%	6.0	5.4	5.6		5.7
	HL:3%	6.0	5.4	5.0		5.5
	LSD:5%	5.3	4.4	4.5		4.7
	FA:5%	5.3	4.0	4.0		4.4
	PC:5%	6.0	4.4	4.5		5.0
SYENITE						
	NONE	6.0	5.6	5.5		5.7
	AS:0.5%	6.2	5.5	6.0		5.9
	HL:2%	5.8	5.1	5.5		5.5
	HL:3%	6.0	5.0	5.0		5.3
	LSD:5%	6.0	4.9	5.0		5.3
	FA:5%	5.5	4.6	5.0		5.0
	PC:5%	6.2	5.1	5.5		5.6
GRAVEL						
	NONE	5.8	5.1	5.5		5.5
	AS:0.5%	6.2	5.1	5.5		5.6
	HL:2%	5.9	4.7	5.3		5.3
	HL:3%	5.7	4.6	5.0		5.1
	LSD:5%	5.6	4.5	4.2		4.8
	FA:5%	5.5	4.2	4.5		4.7
	PC:5%	5.6	4.6	5.1		5.1
NOVACULITE						
	NONE	6.6	5.6	6.5		6.2
	AS:0.5%	6.7	5.6	6.5		6.3
	HL:2%	6.3	5.0	6.2		5.8
	HL:3%	6.3	4.9	5.7		5.6
	LSD:5%	6.0	5.0	5.4		5.5
	FA:5%	6.0	4.6	5.5		5.4
	PC:5%	6.0	4.4	5.0		5.1
LIMESTONE						
	NONE	4.9	4.2	4.0		4.4
	AS:0.5%	4.8	3.7	4.0		4.2
	HL:2%	4.6	3.5	3.9		4.0
	HL:3%	5.0	4.2	3.8		4.3
	LSD:5%	4.5	3.4	3.5		3.8
	FA:5%	4.2	3.3	3.8		3.8
	PC:5%	4.5	3.6	3.5		3.9

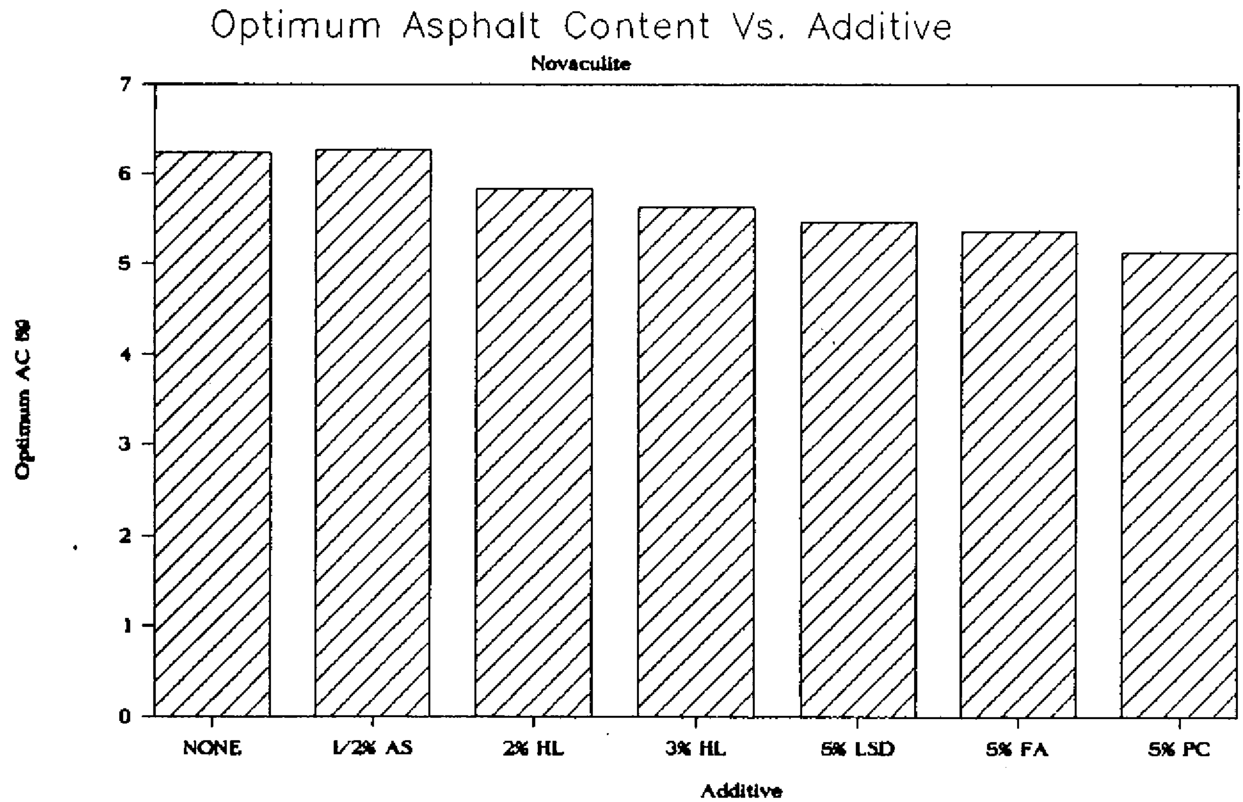

Figure 1. Change in optimum asphalt content with additive for Novaculite

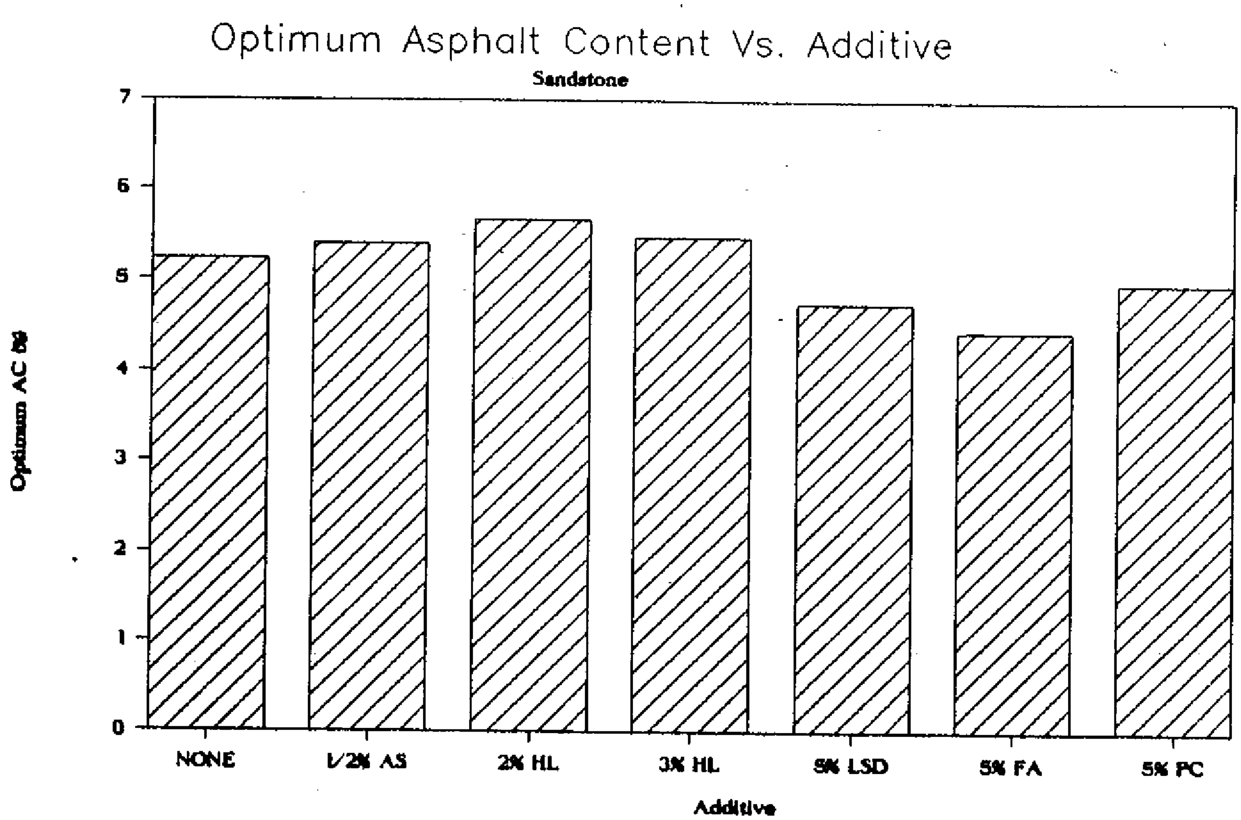

Figure 2. Change in optimum asphalt content with additive for Sandstone

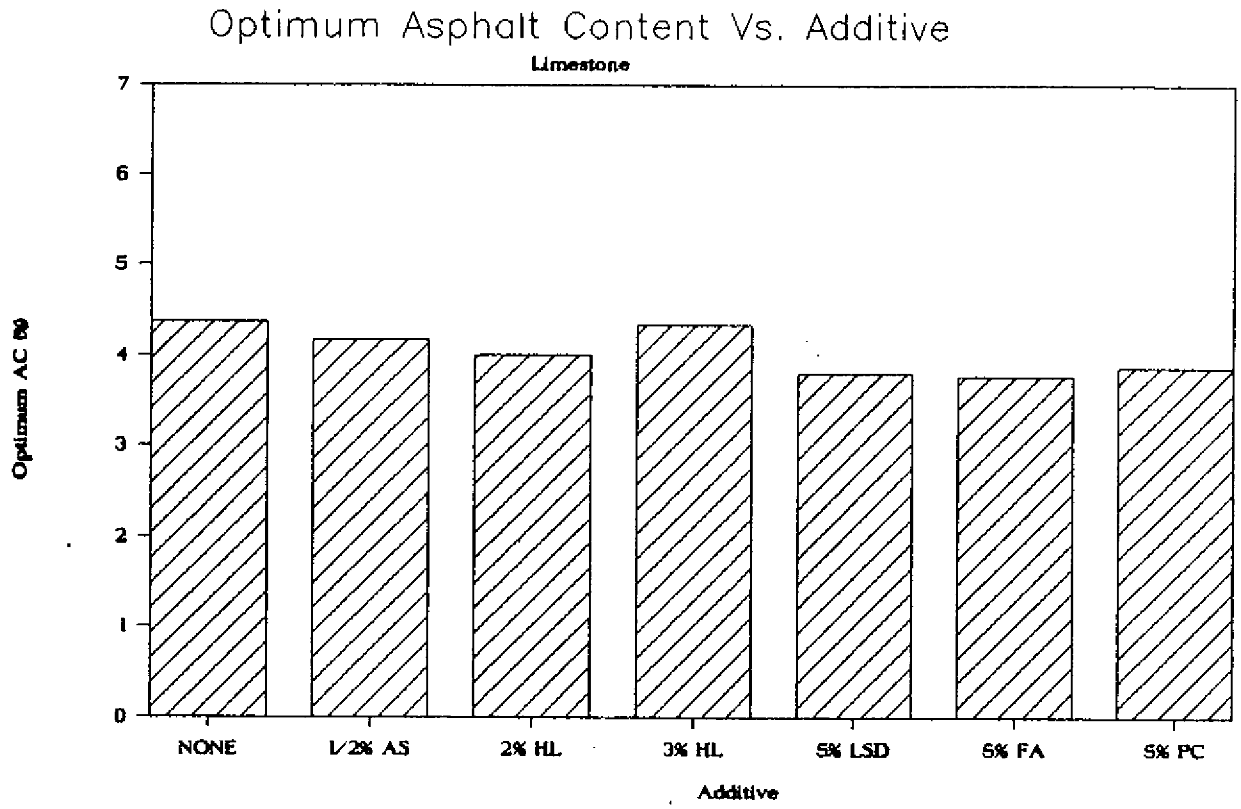

Figure 3. Change in optimum asphalt content with additive for Limestone

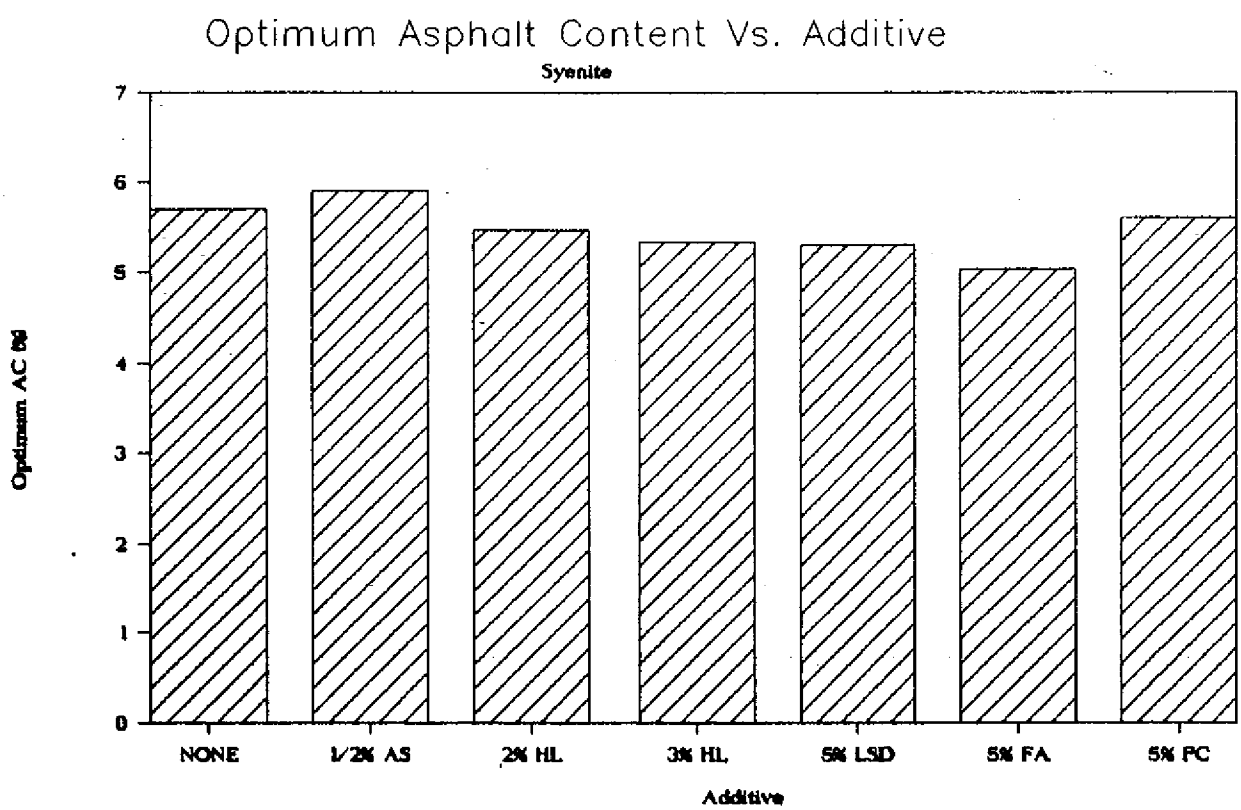

Figure 4. Change in optimum asphalt content with additive for Syenite

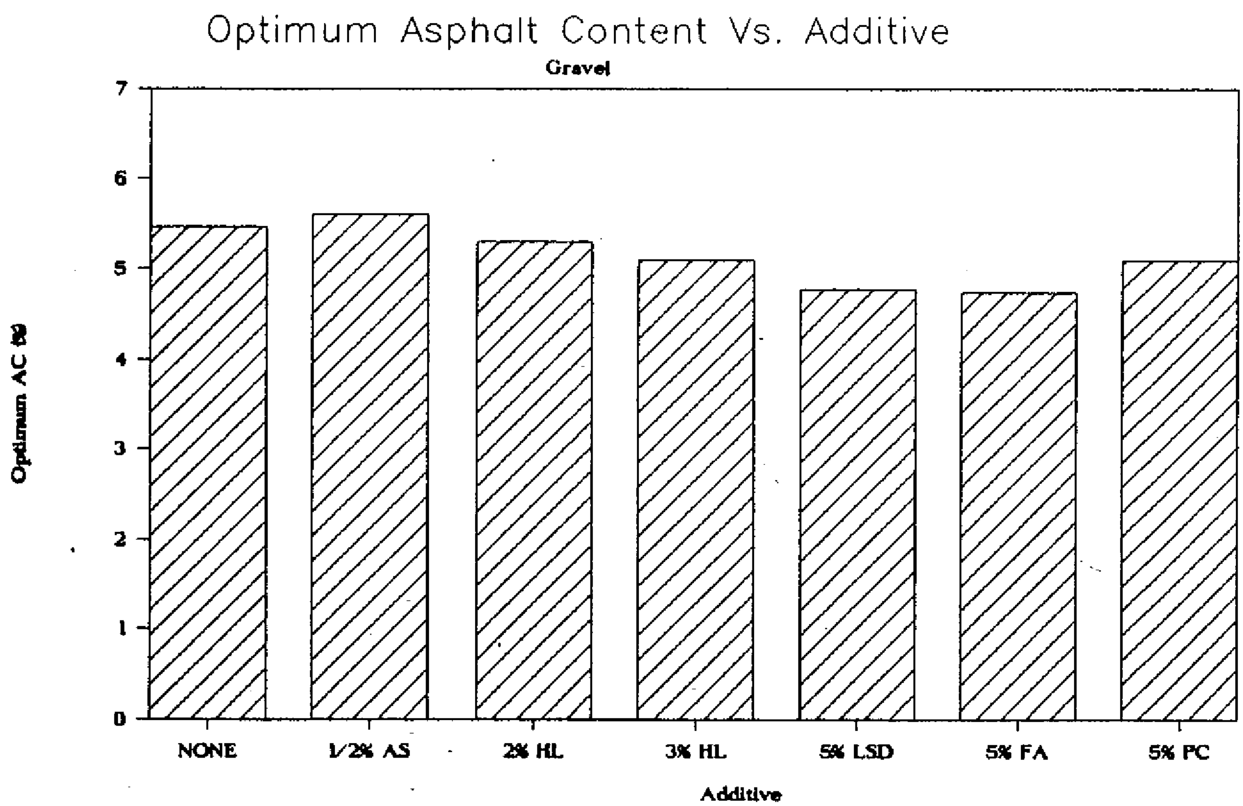

Figure 5. Change in optimum asphalt content with additive for Gravel

TABLE VI

Change in Air Void Content with the Addition of Mineral Fillers

Aggregate	Additive	Air Void Content at 4.5% AC	5.0% AC	5.5% AC	6.0% AC
Sandstone	None	4.7	2.9	1.7	0.7
	AS: 0.5%	-	3.5	1.8	0.8
	HL: 2%	-	5.4	3.7	2.3
	HL: 3%	8.4	5.8	3.7	1.7
	LSD: 5%	3.4	1.3	0.5	0.2
	FA: 5%	2.2	1.0	0.1	0.0
	PC: 5%	3.7	1.3	0.6	0.1
Syneite	None	-	5.9	4.2	2.8
	AS: 0.5%	-	5.7	4.0	1.4
	HL: 2%	-	4.5	2.6	1.6
	HL: 3%	5.7	4.1	2.1	1.2
	LSD: 5%	-	3.6	2.1	1.3
	FA: 5%	4.7	2.3	1.3	0.3
	PC: 5%	-	4.4	2.4	0.9
Gravel	None	-	4.5	3.0	2.0
	AS: 0.5%	-	4.2	2.6	1.1
	HL: 2%	-	3.1	1.7	0.4
	HL: 3%	4.5	2.4	1.3	0.2
	LSD: 5%	4.2	2.3	1.1	0.3
	FA: 5%	3.1	1.6	0.8	0.2
	PC: 5%	4.5	2.3	1.2	0.7
Novaculite	None	-	5.3	4.2	2.9
	AS: 0.5%	-	5.9	4.3	2.6
	HL: 2%	-	4.1	2.7	0.8
	HL: 3%	6.0	3.5	1.8	0.2
	LSD: 5%	-	4.4	2.3	1.1
	FA: 5%	4.3	2.2	0.7	0.0
	PC: 5%	3.8	2.5	1.1	0.0
Limestone	None	2.6	1.3	0.6	
	AS: 0.5%	1.4	0.6	0.0	
	HL: 2%	1.1	0.3	0.0	
	HL: 3%	1.1	0.3	0.0	
	LSD: 5%	1.1	0.2	0.0	
	FA: 5%	0.1	0.0	0.0	
	PC: 5%	0.9	0.1	0.0	

be seen in this table that for a given asphalt content the addition of a mineral filler can substantially lower the air void content. This particularly true of fly ash, portland cement, and limestone dust.

The reduction in voids is not the only design parameter that should be looked at in showing the effect of additives on the optimum AC. The asphalt content found at maximum density and maximum stability should also be addressed. In order to discuss the change in density, the change in VMA with the addition of fillers must be studied. It was pointed out in the literature that mineral fillers do serve to occupy space between the larger particles and contribute toward lowering the VMA. Since a lower VMA results in less asphalt required to produce a mixture with appropriate properties, it can be readily seen that lowering the VMA affects not only density but the other mix properties used in design.

Hydrated lime was found not to be as effective at reducing the design asphalt content of the mixes as the other mineral fillers. However, it is difficult to compare mineral fillers for changes in optimum AC when hydrated lime was added to the mix at 2% and 3% amounts by weight and the other fillers were added at 5% by weight. Of the three fillers added at 5%, fly ash was found to reduce the optimum more followed by limestone dust and portland cement.

An attempt was made to relate the gradation of these fillers with their ability to reduce the optimum AC. The gradations of these fillers are shown in Appendix B. The gradation believed to reduce the optimum asphalt content was one that maintained a well graded curve through 10 to 20 microns and possibly a high percentage of particules smaller than 5 to 10 microns. These properties were believed to extend the gradation down to the lowest practicle degree, thereby filling void spaces to densify the mix and to act as an asphalt extender for those particles finer than 5 to 10 microns. Examination of these gradations show that limestone dust meets these requirements better followed in order by hydrated lime, fly ash, and portland cement. Unfortunately, this order did not coincide with the actual test data even when attempts were made to adjust the hydrated lime results to a 5% addition amount. This may show that other factors such as the type, particle shape, and bulk volume control the effect of mineral filler on the design asphalt content. Likewise, each aggregate type showed different effects from the addition of mineral fillers even though the same aggregate gradation was used.

The properties of the mixes at optimum AC were also studied. These properties are shown in Table VII. It should be noted that optimum AC is defined as the average asphalt content found at maximum stability and density, and 4% air voids for this study only. An actual design would require the mix to meet the air void range of 3 to 5%. Therefore, the optimum would be changed to meet this requirement. No such adjustment is made here.

The water sensitivity of the specimens were evaluated by the percent retained stability and retained stability. It was found that, at optimum AC, portland cement had the highest average percent retained stability followed by fly ash, 3% hydrated lime, 2% hydrated lime, no additive, antistrip agent, and limestone dust. However, if the results in Table VII are carefully studied it can be seen that some of these additives significantly increase the dry stability possibly causing a low percent retained stability. For example, the addition of 3% hydrated lime increased the dry strength by an average of 535 lbs. Even though the wet strengths were also much higher the resulting ratio was lower than other additives that did not produce a high increase in dry stability. The retained stability may be an important indicator of moisture susceptibility along with the percent retained stability.

TABLE VII

Mix Properties at Optimum Asphalt Content

AGGREGATE	ADDITIVE	VALUES AT OPTIMUM STAB (LBS)	AIR VOIDS	RETAINED STAB	VMA	DENSITY	% RET. STAB
SANDSTONE							
	NONE	2600	2.4	1600	14.2	146.0	61.5
	AS:0.5%	2500	2.1	1900	13.7	145.6	76.0
	HL:2%	3750	2.9	2800	16.4	144.4	74.7
	HL:3%	3325	3.7	2750	15.4	143.8	82.7
	LSD:5%	3100	2.4	1400	12.9	147.0	45.2
	FA:5%	2525	2.5	1850	13.6	147.0	73.3
	PC:5%	2500	1.3	2600	12.8	148.4	104.0
SYENITE							
	NONE	1890	3.6	1720	16.7	143.9	91.0
	AS:0.5%	2190	1.7	1860	14.6	146.4	84.9
	HL:2%	2360	2.6	1940	15.2	146.2	82.2
	HL:3%	2510	2.8	2220	14.8	146.4	88.4
	LSD:5%	2200	2.6	1680	14.7	146.9	76.4
	FA:5%	1930	2.3	1740	15.7	147.5	90.2
	PC:5%	2050	2.0	2030	15.1	147.8	99.0
GRAVEL							
	NONE	1800	3.0	1080	15.1	143.6	60.0
	AS:0.5%	2060	2.3	1280	15.0	144.3	62.1
	HL:2%	1950	2.0	1750	14.0	145.4	89.7
	HL:3%	2310	2.3	1900	13.6	145.8	82.3
	LSD:5%	1990	3.1	1300	13.9	146.0	65.3
	FA:5%	1770	2.5	1800	11.3	146.9	101.7
	PC:5%	2120	2.1	1960	14.0	147.6	92.5
NOVACULITE							
	NONE	2120	2.1	1390	15.8	142.8	65.6
	AS:0.5%	2000	1.5	1180	16.1	142.8	59.0
	HL:2%	2140	1.5	2040	14.4	142.7	95.3
	HL:3%	2660	1.5	2110	14.2	143.4	79.3
	LSD:5%	2190	2.3	1640	14.8	145.1	74.9
	FA:5%	2080	1.0	1520	13.3	144.9	73.1
	PC:5%	2280	2.3	1940	11.1	144.3	85.1
LIMESTONE							
	NONE	1960	3.0	1080	13.8	156.1	55.1
	AS:0.5%	2190	4.2	1050	15.5	156.1	47.9
	HL:2%	2500	2.2	1600	11.6	155.8	64.0
	HL:3%	2240	3.7	1520	13.7	152.4	67.9
	LSD:5%	2420	2.2	720	8.6	156.1	29.8
	FA:5%	2010	1.4	1730	10.6	157.6	86.1
	PC:5%	2280	2.8	1940	12.1	156.7	85.1

The samples with 3% hydrated lime were found to have the highest retained stability followed by portland cement, 2% hydrated lime, fly ash, antistrip agent, limestone dust, and no additive. If just the percent retained stability was looked at and not the retained stability, fly ash would be the most effective, however, fly ash is fourth in terms of the retained stability.

The argument can be made that evaluating these additives for moisture susceptibility at optimum (as defined in this report) may not represent true behavior since some of the mix properties are so different. Some air void contents are less than 1.5%. Also VMA values of some of these samples are exceedingly low. Because of these problems, the moisture susceptibility will be looked at with other controlling criteria.

The mix properties were evaluated at asphalt contents designed to produce 2%, 3%, 4%, and 5% air voids. These properties are shown in Tables VIII, IX, X, and XI. In general, it was found that the design air voids had little effect on the retained stability ratio. An additive that was the most effective at 2% designed air voids generally maintained its effectiveness at 3%, 4%, and 5% air void content. In only one instance did the effectiveness change markedly from one air void content to another. There were a few interesting trends found from these tables. The trends are:

1. The antistrip agent used in the mix comparisons (permatac-plus) was very effective with the sandstone and syenite aggregates and compared well with the mineral fillers tested. However, the antistrip performed poorly with limestone and gravel aggregates.

2. Limestone dust compared poorly against the other additives with every aggregate tested. Often, the no antistrip samples were more moisture resistant than those with 5% limestone dust added.

3. The fly ash mineral filler performed well with gravel and limestone in terms of retained stability and retained stability ratio. It performed moderately with the sandstone, syenite, and novaculite.

4. Hydrated lime ranked as one of the better performers in terms of retained stability with every aggregate except syenite. Even then the retained stability ratio was over 80%. Also, no particular percentage of hydrated lime consistently outperformed the other. Sometimes the 2% hydrated lime was superior over the 3% while many times the reverse was found.

Table VIII

Mix Properties at 2% Air Void Design

AGGREGATE	ADDITIVE	ASPHALT CONTENT	VALUES AT 2% AIR VOIDS STAB (LBS)	RETAINED STAB	VMA (%)	DENSITY (PCF)	% RET. STAB
SANDSTONE							
	NONE	5.3	2600	2350	14.2	146.1	90.4
	AS:0.5%	5.4	2500	2675	13.7	145.6	107.0
	HL:2%	6.2	3250	2800	16.7	144.4	86.2
	HL:3%	5.9	3000	2750	15.2	144.8	91.7
	LSD:5%	4.8	3000	1400	12.8	147.2	46.7
	FA:5%	4.6	2400	1850	12.6	147.2	77.1
	PC:5%	4.8	2800	2500	13	148.2	89.3
SYENITE							
	NONE	6.5	1600	1500	16.9	144.6	93.8
	AS:0.5%	5.8	2170	1940	14.7	146.2	89.4
	HL:2%	5.7	2330	1920	15.2	146.4	82.4
	HL:3%	5.5	2440	2140	14.7	146.8	87.7
	LSD:5%	5.5	2140	1700	14.7	147.2	79.4
	FA:5%	5.1	1930	1720	15.8	147.6	89.1
	PC:5%	5.6	2050	2030	15	147.8	99.0
GRAVEL							
	NONE	6	1630	1210	15.5	143.8	74.2
	AS:0.5%	5.7	2020	1260	14.9	144.5	62.4
	HL:2%	5.3	1940	1760	14	145.4	90.7
	HL:3%	5.1	1920	1900	13.6	145.8	99.0
	LSD:5%	5.1	1880	1380	13.6	146.5	73.4
	FA:5%	4.9	1760	1800	11.3	147.1	102.3
	PC:5%	5.1	2120	1960	14	147.6	92.5
NOVACULITE							
	NONE	6.2	2120	1390	15.8	142.8	65.6
	AS:0.5%	6.1	1870	1250	16.2	142.4	66.8
	HL:2%	5.7	2130	2000	14.5	142.3	93.9
	HL:3%	5.4	2620	2090	14.3	143	79.8
	LSD:5%	5.6	2070	1660	14.8	145.3	80.2
	FA:5%	5.1	2000	1340	13.3	143.6	67.0
	PC:5%	5.2	2270	1950	11.1	144.5	85.9
LIMESTONE							
	NONE	4.7	1900	980	13.6	156.5	51.6
	AS:0.5%	4.3	2170	1280	15.3	156.3	59.0
	HL:2%	4.1	2490	1710	11.6	155.9	68.7
	HL:3%	4.7	2060	1750	13.1	156.1	85.0
	LSD:5%	3.9	2360	980	8.5	156.4	41.5
	FA:5%	3.6	2000	1750	10.9	157	87.5
	PC:5%	4.1	2160	1940	11.9	157.3	89.8

Table IX

Mix Properties at 3% Air Void Design

AGGREGATE	ADDITIVE	VALUES AT 3% AIR VOIDS ASPHALT CONTENT	STAB (LBS)	RETAINED STAB	VMA (%)	DENSITY (PCF)	% RET. STAB
SANDSTONE							
	NONE	5	2825	1600	14.2	145.6	56.6
	0.5% AS	5.1	2450	2000	13.8	145.0	81.6
	2% HL	5.7	3750	2800	16.4	144.4	74.7
	3% HL	5.7	3150	2800	15.2	144.4	88.9
	5% LSD	4.6	2600	1450	13.1	146.5	55.8
	5% FA	4.3	2600	1850	12.6	146.8	71.2
	5% PC	4.7	3050	2400	13.3	147.8	78.7
SYENITE							
	NONE	5.8	1840	1380	16.7	144.2	75.0
	0.5% AS	5.6	2090	2060	15.0	145.2	98.6
	2% HL	5.4	2350	1960	15.3	146.0	83.4
	3% HL	5.3	2510	2220	15.8	146.3	88.4
	5% LSD	5.2	2220	1610	14.7	146.7	72.5
	5% FA	4.8	1920	1740	15.5	147.1	90.6
	5% PC	5.4	2050	2080	15.2	147.3	101.5
GRAVEL							
	NONE	5.5	1800	1080	15.1	143.6	60.0
	0.5% AS	5.4	2060	1300	15.0	143.9	63.1
	2% HL	5.0	1880	1680	14.3	144.3	89.4
	3% HL	4.9	2320	1880	13.7	145.1	81.0
	5% LSD	4.8	1990	1300	13.9	146.0	65.3
	5% FA	4.5	1780	1480	11.4	146.6	83.1
	5% PC	4.8	2080	1900	14.1	147.1	91.3
NOVACULITE							
	NONE	6.0	1940	1350	16.1	142.2	69.6
	0.5% AS	5.9	1820	1320	16.3	141.7	72.5
	2% HL	5.4	2110	1820	14.8	141.5	86.3
	3% HL	5.1	2480	1980	14.5	142.1	79.8
	5% LSD	5.2	2180	1500	14.9	144.4	68.8
	5% FA	4.8	1900	1240	13.5	143.9	65.3
	5% PC	4.9	2160	1960	11.2	143.8	90.7
LIMESTONE							
	NONE	4.4	1970	1080	13.8	156.1	54.8
	0.5% AS	4.0	2200	920	15.7	155.3	41.8
	2% HL	3.7	2480	1340	11.7	155.2	54.0
	3% HL	4.5	2180	1680	13.4	153.3	77.1
	5% LSD	3.6	2480	500	8.8	155.4	20.2
	5% FA	3.5	1980	1740	11.2	156.2	87.9
	5% PC	3.9	2280	1930	12.1	156.7	84.6

Table X

Mix Properties at 4% Air Void Design

AGGREGATE	ADDITIVE	ASPHALT CONTENT	VALUES AT 4% AIR VOIDS STAB (LBS)	RETAINED STAB	VMA (%)	DENSITY (PCF)	% RET. STAB
SANDSTONE							
	NONE	4.7	2730	1600	14.5	144.6	58.6
	0.5% AS	4.9	2350	2100	13.8	144.6	89.4
	2% HL	5.4	3850	2500	16.4	144.1	64.9
	3% HL	5.4	3500	2680	15.4	143.6	76.6
	5% LSD	4.4	3170	1620	13.5	145.3	51.1
	5% FA	4.0	2740	1780	12.9	146.0	65.0
	5% PC	4.4	3130	2200	13.6	147.0	70.3
SYENITE							
	NONE	5.6	1900	1280	16.7	143.7	67.4
	0.5% AS	5.5	2020	2080	15.1	144.0	103.0
	2% HL	5.1	2260	2000	15.4	145.1	88.5
	3% HL	5.0	2560	2340	15.1	145.3	91.4
	5% LSD	4.9	2250	1460	14.9	146.1	64.9
	5% FA	4.6	1870	1680	15.1	146.5	89.8
	5% PC	5.1	1990	2110	15.5	146.2	106.0
GRAVEL							
	NONE	5.1	1670	1080	15.4	142.6	64.7
	0.5% AS	5.1	1900	1260	15.2	143.1	66.3
	2% HL	4.7	--	--	--	--	--
	3% HL	4.6	2260	1730	14.1	143.7	76.5
	5% LSD	4.5	2070	1120	14.2	144.9	54.1
	5% FA	4.2	1780	1830	11.6	145.9	102.8
	5% PC	4.6	1980	1850	14.3	146.3	93.4
NOVACULITE							
	NONE	5.6	1730	1170	16.1	141.3	67.6
	0.5% AS	5.6	1790	1340	16.4	140.9	74.9
	2% HL	5.0	2100	1580	15.0	140.7	75.2
	3% HL	4.9	2160	1980	14.6	141.6	91.7
	5% LSD	5.0	2170	960	15.4	143.1	44.2
	5% FA	4.6	1760	1100	13.6	143.1	62.5
	5% PC	4.4	1880	1900	11.3	142.9	101.1
LIMESTONE							
	NONE	4.2	2000	940	14.1	155.7	47.0
	0.5% AS	3.7	2180	930	16.1	154.1	42.7
	2% HL	3.5	2420	1220	12.2	153.7	50.4
	3% HL	4.2	2260	1440	13.8	152.1	63.7
	5% LSD	3.4	2480	400	9.1	154.4	16.1
	5% FA	3.3	--	--	--	--	--
	5% PC	3.6	2350	1900	12.6	155.3	80.9

Table XI

Mix Properties at 5% Air Void Design

AGGREGATE	ADDITIVE	VALUES AT 5% AIR VOIDS ASPHALT CONTENT	STAB (LBS)	RETAINED STAB	VMA (%)	DENSITY (PCF)	% RET. STAB
SANDSTONE							
	NONE	4.4	2100	1600	15.0	143.4	76.2
	0.5% AS	--	--	--	--	--	--
	2% HL	5.1	3820	1950	16.5	143.6	51.0
	3% HL	5.2	3900	2500	15.7	146.0	64.1
	5% LSD	4.0	2880	2650	14.2	143.2	92.0
	5% FA	3.8	2750	1800	13.3	145.1	65.5
	5% PC	4.1	3040	1750	14.2	146.0	57.6
SYENITE							
	NONE	5.2	1860	1180	16.9	142.9	63.4
	0.5% AS	5.2	1880	1700	15.5	143.1	90.4
	2% HL	--	--	--	--	--	--
	3% HL	4.8	2540	2420	15.3	144.6	95.3
	5% LSD	--	--	--	--	--	--
	5% FA	--	--	--	--	--	--
	5% PC	--	--	--	--	--	--
GRAVEL							
	NONE	--	--	--	--	--	--
	0.5% AS	--	--	--	--	--	--
	2% HL	--	--	--	--	--	--
	3% HL	--	--	--	--	--	--
	5% LSD	4.5	2070	1120	14.2	144.9	54.1
	5% FA	--	--	--	--	--	--
	5% PC	--	--	--	--	--	--
NOVACULITE							
	NONE	5.1	1600	740	16.1	140.5	46.3
	0.5% AS	5.3	1780	1260	16.6	140.4	70.8
	2% HL	--	--	--	--	--	--
	3% HL	4.9	2160	1980	14.6	141.6	91.7
	5% LSD	--	--	--	--	--	--
	5% FA	--	--	--	--	--	--
	5% PC	--	--	--	--	--	--
LIMESTONE							
	NONE	3.9	2010	660	14.4	155.0	32.8
	0.5% AS	3.5	2120	1200	16.2	153.3	56.6
	2% HL	--	--	--	--	--	--
	3% HL	4.0	2290	1300	14.1	151.4	56.8
	5% LSD	3.2	2460	340	9.6	153.2	13.8
	5% FA	--	--	--	--	--	--
	5% PC	--	--	--	--	--	--

5. Portland cement performed better overall in terms of retained stability ratio. Portland cement worked well with all the materials tested.

The change in retained stability ratio and retained stability with design air void content for each mineral filler/antistrip/aggregate combination was studied to determine if a correlation exists. No relationship was found between air void content and retained stability ratio in contrast to that reported earlier by Ford (12). However, the results in this report do not necessarily contradict Ford's earlier findings. Dr. Ford used samples with air void contents that were varied by different compactive efforts. The air voids produced in this study were varied by adjusting the asphalt content while holding the compaction effort constant. It can easily be seen that these methods of varying the void content are entirely different and may not produce the same results. In varying the compactive effort, air voids are adjusted by varying the degree of contact between aggregate particles producing more void spaces. Air void contents varied by adjusting asphalt content are produced by adjustments in film thickness around the aggregate particles causing the separation or densification of the void spaces between the particles. Because of the differences the air void content adjusted by varying the asphalt content (the Marshall procedure) should be distinguished by refining these as the design air void content. In fact Tables VIII thru XI show the mix properties of samples desinged for 2%, 3%, 4%, and 5% air voids. This is very different from designed mixes that are altered by varying the compaction effort.

There are several other means of comparing the moisture susceptibility of the asphalt mixes as measured by the immersion compression test. The mixes can be evaluated by VMA, asphalt content, and maximum density. However, these comparisons may be of little value because of the great change in the mix properties due to the addition of the mineral fillers. In many instances, the addition of a filler resulted in a mix with undesirable Marshall mix properties requiring other adjustments to be made. Unfortunately, holding the gradation constant to reduce the number of samples resulted in producing mixes that did not meet void requirements but could have with adjustments in the gradation. This problem can be easily illustrated by looking at the design curves in Appendix B. For the limestone mixes, the addition of 5% fly ash and limestone dust reduced the VMA to an unacceptable level. Mixes with a VMA this low are usually deficient in binder or air voids. There simply is no room for the asphalt. Likewise, fly ash was found to reduce the VMA excessively for the gravel mixes and portland cement for the novaculite mixes. An adjustment in aggregate grading may have allowed voids to be built into the mix for satisfactory compliance.

Chapter V

CONCLUSIONS AND RECOMMENDATIONS

On the basis of the experimental work covered by this report and within the limitations of the test procedures and materials utilized in this investigation, the following conclusions are made:

1. The coating ability of an antistrip agent, as measured by the boil test, can be affected by the source or type of asphalt cement.

2. The addition of mineral filler can significantly alter the optimum asphalt content of a mixture. The addition of the mineral fillers tested in this report in concentration of 2%, 3%, or 5% were found to reduce the optimum asphalt content. Fly ash was found to affect the optimum AC the most of all the fillers tested.

3. The water susceptibility, as measured by the immersion compression test, of the mixes was improved by the use of mineral fillers and/or antistrip agent. However, behavior varied with aggregate source. In some cases, one mineral filler would provide good performance with one aggregate source and not another.

4. The antistrip agent used in the mix comparisons (permatac plus) was very effective with the sandstone and syenite aggregates and compared well with the mineral fillers tested. However, the antistrip performed poorly with limestone and gravel aggregates with the concentration of antistrip used in these tests.

5. Limestone dust compared poorly in relationship with the other additives with every aggregate tested. However, it should be mentioned that this poor performance is with one gradation and one additive percentage.

6. Hydrated lime added at 2% and 3% by weight to the mix was found not to be overwhelmingly superior in terms of retained stability to other mineral fillers tested as first believed. Other mineral fillers compared well with hydrated lime.

RECOMMENDATIONS

1. The boil test results showed that the results varied with the type or grade of asphalt cement. The boil test is currently used by the Arkansas State Highway and Transportation Department to qualify anti-strip agents for possible use in Arkansas. Since the boil test is used for only one asphalt and one aggregate, the qualified antistrip agents may be only effective with one asphalt and one aggregate. While the argument can be made that this is adequate because the effectiveness is checked in design with the immersion compression test, it may be beneficial to expand our prequalification by broadening the boil test to include other aggregates and asphalts. This would give the added benefit of having two stripping tests for a designed mix.

2. The amount of mineral filler used in mix should be allowed to vary to produce a mix with appropriate void requirements. It was found that the addition of 5% limestone dust reduced the voids to unacceptable levels in many instances. A reduction in the amount of limestone dust may have satisfied the void requirements and produced a mix with better moisture resistance.

3. Futher research in the effects of using hydrated lime as a mineral filler is needed. The results of these tests showed that the hydrated lime was not as superior to other fillers as reported in the Literature. Also, most states use approximately 1% hydrated lime. No research was conducted in this report with hydrated lime added at 1%.

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