DEVELOPMENT OF LABORATORY MIX DESIGN PROCEDURE AND DETERMINATION OF THICKNESS EQUIVALENCY FACTORS FOR SOME OKLAHOMA FOAMIXES

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CHAPTER I

INTRODUCTION

Statement of the Problem

In many areas, the lack of availability and the increased cost of high quality aggregates has created a need to use locally accessible materials in base courses for road construction. Frequently, these materials are of inferior quality and must be treated or stabilized to improve their properties and make them suitable for incorporation in a pavement structure. Asphalt has been a common base stabilizer for many years; however, the requirements for materials selection and mix design and construction techniques for bases have relied heavily on criteria developed for asphalt concrete surface course mixes. Thus, because of stricter requirements, materials and construction techniques are being utilized which significantly increase the cost and provide a stabilized material whose properties may be in excess of those actually required by traffic and environment.

In recent years, a relatively new and economical stabilizing agent has been developed. This agent, "foamed asphalt", is an advance in the use of bitumenous materials for road construction purposes. The asphalt foaming process was first proposed by Csanyi (1, 2, 3) in the mid 1950's. The original process consisted of introducing steam into hot asphalt cement as it passes through a specially designed nozzle such that the asphalt cement was ejected as a foam. Due to the inconvenience of

Csanyi's process, the comparatively low cost of asphalt and energy and the availability of quality aggregate, in the 1950s, foamed asphalt was not widely used for about fifteen years.

In 1968, Mobil Oil of Australia developed some improvements for the production of foamed asphalt. The most important improvements were the use of cold water with hot asphalt cement and a modified foaming nozzle to produce the foamed asphalt (4). The Continental Oil Company has further developed the process and has been licensed by Mobil Oil of Australia to market the process in the United States.

Other recent advancements that improve asphalt foam quality and the installation of field projects employing foamed asphalt mixtures which have provided experience and enhanced progress in the construction procedures used. In the past 13 years the asphalt foaming process has been successfully used in Australia and more recently in South Africa and several American states for the stabilization of marginal quality pavement materials.

The major problem with using foamed asphalt mixes in Oklahoma is that basic design criteria for such mixtures as well as thickness design considerations are not available. These criteria will have to be developed and must be based on the use of locally available materials--aggregates as well as asphalts.

Method and Scope of Study

The first objective of this research was to conduct a laboratory investigation to examine the applicability of foamed asphalt mixtures made with Oklahoma fine aggregates and to explore some characteristics of such mixtures that affect the interactions between the components and the quality of the material. After establishing the desirable characteristics and properties of Oklahoma foamixes, these findings can provide some guidelines for the development of a foamed asphalt mix design procedure that is suitable for materials and environmental conditions indigenous to Oklahoma.

The second objective was to evaluate these foamixes in terms of their ability to perform as part of a structural pavement system. This study was considered necessary in order to establish a mechanistic thickness design method for these mixtures, based on average climatic conditions in Oklahoma.

The experimental phase of the study included the selection and analysis of five different marginal quality aggregates and aggregate blends and an asphalt cement. Considerable experience in the mechanics of foamixing was gained before mixing, molding and subsequently evaluating the foamed asphalt mixtures. The evaluative testing program was designed so that a variety of conventional testing methods was employed. Several of the tests performed throughout the program were modified because of the atypical characteristics of foamed asphalt mixtures.

The effects of temperature changes and moisture intrusion on the strength of foamixes were considered. A comparison between the stabilities of foamixes and hot mixes prepared from the same materials was included. Since foamixes are classified as cold type mixtures, the effect of strength gain due to moisture loss by curing was also studied.

A computer program designed for multi-layered system analysis was employed in the analytical phase of the study. For specific design parameters, the thicknesses of foamixes were calculated and compared to those required for the high quality asphalt concrete used on the AASHTO Road test pavement. Thickness equivalency ratios for four different foamixes were determined based on the changes in both pavement temperature and subgrade moisture throughout the year. To cover a wide range of thickness design requirements, the thickness equivalency ratios were computed at several levels of the number of equivalent single axle load repetitions to failure and over several different subgrade strengths.

CHAPTER II

LITERATURE REVIEW

In the preparation of plant mix hot bituminous paving mixtures, with asphalt cement as the binder, the viscosity of the binder is adjusted and controlled by the temperature of the binder and the aggregates. A relatively high temperature must be maintained in this type of mix during transportation and laying to assure desired plasticity for spreading and compacting. The heating of the binder to adjust its viscosity and heating of the aggregates to dry them and maintain the temperature of the mix are costly. Such pavements are therefore used generally for high type heavily traveled pavements.

When such high type mixes are not needed or cannot be afforded, other binders such as cut-back asphalts or emulsified asphalts, can be used as the binder. Since the viscosity of a cut-back asphalt is adjusted by the type and quantity of solvent added to the base asphalt cement, the need for further adjustment of viscosity of the binder by heat during mixing is reduced. The viscosity of an emulsified asphalt is adjusted by the method used in preparing the emulsion, by the type of emulsifier used, and by the quantity of water added. A wide variety of grades of emulsified asphalts are available to meet the needs of various applications. When an emulsified asphalt is used in a bituminous mix as the binder, the need for heating the binder is practically eliminated. Furthermore, by virtue of the emulsifying agent used in this type of binder, good adhesion, even with cold and damp aggregates, can be attained.

Although production of a bituminous mix is simplified when these materials are used as the binder, laying procedures become more complicated. The binder should set rapidly after the spreading of the mix to permit proper compaction, i.e., the solvent in a cut-back asphalt should be removed or the water released from an emulsion should be eliminated. For that reason, mixes using these materials as the binder must be aerated to permit the evaporation of either the solvent or the water. This not only requires additional construction equipment and operations, but it also delays the opening of the facility to traffic. Although the use of these materials as binders improves one phase of the procedure, it falls short in another.

An ideal binder for use in low cost paving mixtures would be one whose physical properties, i.e., viscosity, surface tension etc., were such that when mixed with cold and even damp aggregates it would retain the desired plasticity of the mixture until it was spread and compacted, and then set quickly. When hot asphalt cement is combined with steam or water, the mixture foams much like an agitated detergent. This "foamed" asphalt has many of the aforementioned characteristics of an ideal binder material.

Systems of Foaming Asphalt

Csanyi (1, 2, 3) developed the process of foaming asphalt using steam during the mid 50's. In 1968 the patent rights to operate the process were acquired by Mobil Oil of Australia. By 1971 Mobil Oil had developed a new system to foam asphalt cement.

Csanyi's Original System (1957)

In this system, the asphalt cement enters a specially designed nozzle tip where it is foamed instantaneously by saturated steam. The continual flow of the asphalt and steam discharges the foamed asphalt from the nozzle. A spray bar assembly was developed to introduce the asphalt foam into a mixer in the quantity, time, and manner needed for thorough mixing with an aggregate. The general assembly of the foaming system is shown in Figure 1(a).

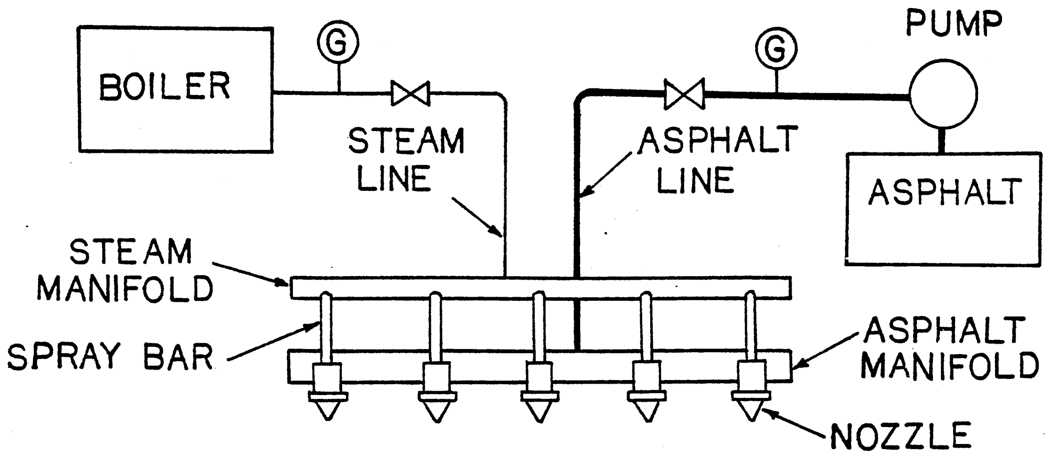
Mobil Oil of Australia Original System (1968)

The original Mobil Oil system was a modification of Csanyi's system with a third manifold for the addition of water. The manifold directly injected water into the steamlines immediately before entry of the steam into the foam nozzle (4).

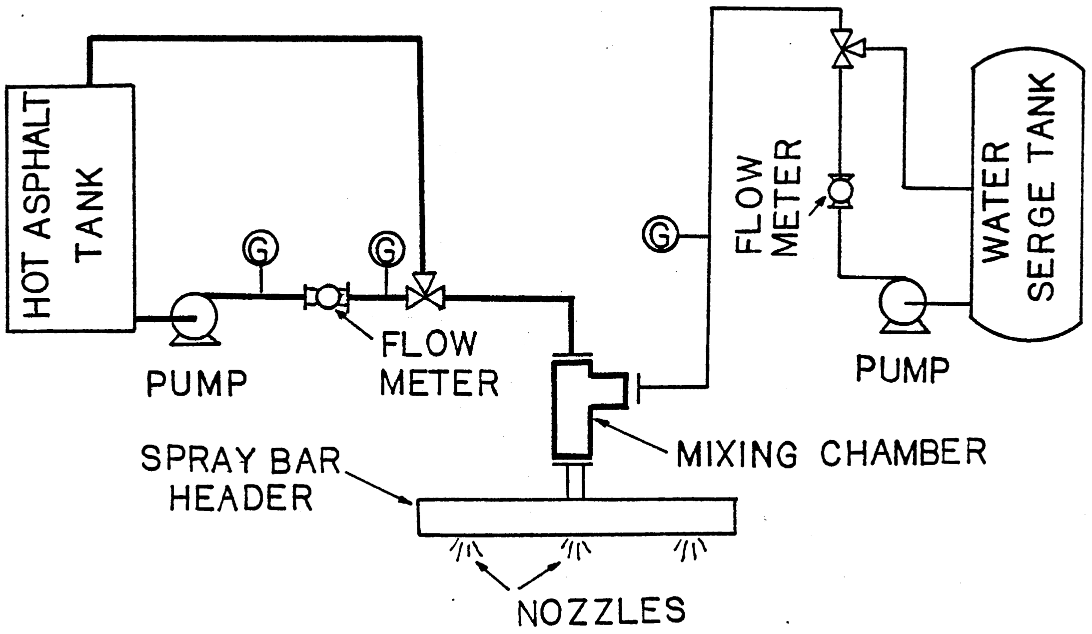
The New Mobil Oil of Australia System (1971)

This system has been used all over the world for foamed asphalt production, since it was developed in 1971. In this system foam is produced in a mixing chamber by introducing water only into the hot asphalt stream in a controlled fashion. The partially expanded foam mixes to a uniform consistency in the mixing chamber and is then delivered to the spraybar, with each nozzle receiving uniform quality foam. This modified process reduced the required number of units as shown in Figure 1(b).

The new Mobil foamed asphalt system has been adapted to continuous mix plants, drum mixers, and batch plants. It has also been used in travel plants for processing in-situ material for soil stabilization work (5).



(a) Csanyi's Original System



(b) System Developed by Mobil Oil of Australia

Figure 1. Foaming Systems

Foamed Asphalt Properties

"Foamed Asphalt" is asphalt cement temporarily changed into a high volume, low effective viscosity foam, having characteristics suited for ready incorporation into damp soil or aggregates. After a period of time, the asphalt cement reverts to its original physical properties (4).

Physical Characteristics

The major physical characteristics of asphalt cement when foamed include the following (1, 4):

1. High volume increase which can be as high as twenty times the original volume of asphalt cement.
2. Low viscosity at atmospheric temperature.
3. Sticky, rubbery and highly ductile frothy material.
4. Regains original properties after foam breaks.

Quality Control Measures

Acceptable asphalt foaming action has been based on determinations made of the expansion level obtained and the stability of the foam. Martin and Bowering (6) suggested that the physical properties of the foamed asphalt which affected the final mix characteristics were the expansion ratio and the half life stability. "Foam Expansion Ratio" is the ratio of the foam volume to that of the original asphalt volume. "Foam Half Life" is the time in seconds between the completion of sample withdrawal and the time when the foamed sample volume has shrunk to a measured one half of its original volume.

The two measures mentioned in the previous paragraph were found to be affected by the amount of water in the foam, the temperature of the asphalt cement and the amount of foam produced (7). An asphalt temperature of 325^{O}F (163^{O}C) and the addition of 1.5 to 2.5 percent water by mass of asphalt have proved to be satisfactory for the production of well expanded and stable foamed asphalt (4, 6, 8, 9, 10, 11, 12).

Foamed Asphalt Additives

Silicone treated asphalt cements are commonly found in the United States (12). Some agencies stipulate the use of silicone as a defoamant during the refining process. To counteract or neutralize the effect of these antifoaming materials an additive is needed to allow the asphalt to foam and sustain the desired foaming action. Asphalt foam additives of approved formulation are available commercially (10, 13). An addition rate of 0.4 to 0.7 percent by mass of asphalt has proved effective in use on asphalts previously treated with customary levels of antifoaming additives (12).

Foamed Asphalt Mixes

A foamed asphalt mixture has been defined as a mixture of wet unheated aggregates and asphalt cement mixed while the asphalt is in a foamed state. In the field, the foam is developed in a foaming chamber and sprayed through special nozzles into a pugmill type mixing chamber where it is mixed with unheated, moist aggregate. This paving mixture can be placed on the roadway with conventional construction equipment.

Based on experience gained in Australia (6), South Africa (8) and Colorado (14), foamed asphalt mixtures appeared to have the following economic, applicational and environmental advantages (10):

1. Cold mix base-course can be produced with cold, wet and marginal aggregates including sand and gravel.
2. No aeration or curing is required before compaction.
3. Conventional equipment can be used with minimum modifications.
4. Cost savings can be achieved if marginal aggregates are locally available or if a binder, e.g., asphalt emulsion, has to be hauled long distances.
5. Less energy consumption as compared with hot mix or stabilization using cutbacks.
6. Minimum pollution problems when compared to stabilizing with asphalt cutbacks.

Properties of Foam Mixes

Foam mixes normally produce fairly stiff, stable mortar type material in which the asphalt is concentrated effectively in the finer fraction of the aggregate, especially the fine sand and silt fraction (6, 8, 10, 11, 15, 16). The type of mix usually obtained is similar to that described, in the literature, by Benson and Becker (17) as a "Phase-Mixture".

Martin and Bowering (6) concluded that for attainment of particular required properties in the soil-asphalt mix, it was neither necessary, nor desirable to achieve virtually uniform asphalt distribution. The selective ability of foamed asphalt to coat the fines forming a mortar between the larger soil particles created a new type of asphalt-aggregate structure which was different from other asphalt mixtures.

In Australia (16), it was found that foamed asphalt, crushed rock specimens were significantly less affected by temperature or loading time than were specimens produced by conventional hot mixing. This was attributed to the greater mechanical interlock of the larger aggregate provided by the strong mortar consisting of asphalt and fines.

Factors Affect the Quality of Mixtures

Some factors have proved to significantly affect the quality and properties of foamed asphalt mixture both as workable mixtures and as finished pavements These factors can be summarized from the literature as follows:

1. Aggregate Quality. A minimum of 3 to 5 percent passing the No. 200 sieve was considered a basic requirement to get a promising foamed asphalt mixture (3, 6, 10, 18, 19). Lee (10), suggested an upper limit of the percent passing the No. 200 sieve in the range of 35 to 40 percent.

2. Mixing Moisture Content. Both Csanyi's original work and recent studies in Australia and the United States (10, 12, 20, 21, 22) have pointed out the need for mixing water in soil-aggregate before adding the foamed asphalt. This mixing water was needed to soften the heavy soil fraction so that the agglomorations could be broken up and uniformly distributed through the mix. It also separates the fine particles and suspends them in a liquid medium, making channels of moisture through which foamed asphalt may penetrate to coat all particles (3).

In most literature, the amount of water required to get an optimum foamed asphalt mix was considered an important argument that needed to be considered. Csanyi suggested that the proper amount of water for any mix might be determined by a few trial batches. In recent work in Australia, Bowering (23) suggested that the optimum mixing water content should be the "Fluff Point", which is the moisture content at which the soil-aggregate has its maximum bulk volume. Use of the "Fluff Point" as the mixing moisture content was also recommended by Wood (11) and by Epps (9). Lee (10) found that optimum mixing moisture content varied with gradation of aggregate (particularly the percent passing the No. 200 sieve), and ranged from 65 percent to 85 percent of a soil's optimum moisture content as determined by standard AASHTO test. A term called "Total Fluid Content", defined as the mixing water plus foamed asphalt in a mix, has recently been proposed (19). It was recommended that the total fluid content might be equal to the standard optimum moisture content to achieve maximum compaction of the soil-aggregate mixture.

3. Asphalt Content. As with any asphalt-aggregate mix, the structural properties of foamed asphalt mixtures are dependent on the level of asphalt content. In a study in Australia (6), it was found that mixes made with foamed asphalt and with asphalt emulsion had similar properties up to an asphalt content level of 1.5 percent by dry weight of aggregate. Above this level of asphalt content the foam mix displayed improved structural properties.

As mentioned in the previous section, foamed asphalt had a selective ability to coat the fine fraction of a soil-aggregate. Based on this phenomenon, it can be predicted that foamed asphalt mixtures will gain their peak stability at lower asphalt content levels than other conventional asphalt mixtures. It is also anticipated that the range of optimum asphalt content will be more narrow than in other asphalt mixtures.

4. Quality of Asphalt Foams. For soil-aggregate stabilization, the recommended foamed asphalt quality controls were expansion ratios between 8 and 15 with a minimum of 25 seconds half-life (6, 10). In a comprehensive experiment to reveal the effects of foam expansion ratio and half-life on the characteristics of foamed asphalt mixtures, Lee (10) stated that: Within a half-life of 10 seconds to 140 seconds and a foam ratio of 5 to 20, no differences could be detected in the properties of resulting foam mixes.

5. Curing Condition. Although foamed asphalt cold mixes do not have the curing problems associated with cutback or asphalt emulsion mixes (1, 2, 3), the curing condition must be considered in the mix design and evaluation. Some premix moisture is always required for the best workability and coating of soil particles. Experience has indicated that cold wet foamed asphalt mixes tend to improve with age, traffic and temperature (16). These conditions contribute to the removal of moisture from the compacted mix.

Two curing conditions were used by Csanyi (10): an air cure at room temperature for three days for mixes to be laid in cool weather and a warm cure at 120°F (49°C) for three days for mixes to be laid in warm weather.

Bowering (24), suggested that specimens be oven cured while in the mold for three days at 140°F (60°C) prior to testing. Abel (14), used three types of curing conditions: three days at room temperature, one day at 140°F (60°C) and three days at 140°F (60°C). Studies in Australia (16) showed that for a curing temperature range of 68 to 140°F (20 to 60°C), the major factor affecting the test results was the moisture content to which specimens had been cured. Lee's work confirmed these findings (10). Epps (9), cured the test specimens out of the mold for 72 hours at room temperature followed by 4 days of vacuum dessication. Tentative but unpublished data (12) suggested that curing temperatures above 120^{o}F (49^{o}C) seemed to seal the surface of some specimens which then developed internal pressures that distorted and damaged them.

The choice of curing the specimens in or out of the mold, influences the condition of the specimens (10). As cold mixes can initially be quite fragile, a specimen curing procedure was developed at Purdue University (11). In this procedure, specimens were cured initially in the mold followed by an out of mold cure period. Lee (10) found that the Marshal stability was influenced more by whether the specimens were cured in or out of the mold than by curing temperature, if curing times were adjusted. He concluded that, for the same curing time and temperature, specimens cured out of the mold had higher Marshal stability values than those cured in the mold.

Ruckel, Acott and Bowering (18) proposed a curing method which suggested how molded foamed asphalt specimens could be treated to simulate the condition in the field after short, medium and long cure periods.

Mix Design Procedures

Under road conditions, foamed asphalt mixtures have some similarity to materials which incorporate emulsion or cutback binders. They each require curing in the field prior to reaching their ultimate strength. As the asphalt is applied in a foamed condition the mix preparation method does, however, differ from those used for emulsion and/or cutback binders. The applicability of some standard test procedures to evaluate both foamed asphalt and other cold mixes has been studied previously by

Mobil Oil of Australia (8, 24).

The need for a standard laboratory procedure for preparation of design mixes using foamed asphalt and treatment of test specimens to prepare them for evaluation tests has resulted in many mix design proposals (10, 12, 19, 25). Some of these mix design procedures will be summarized in the following paragraphs.

Mobil Oil of Australia Procedure

This procedure was first derived after considerable experience in testing a variety of soils stabilized by foamed asphalt produced by the old saturated steam system. A sequence of tests were conducted, on a routine basis, on samples of soils submitted for evaluation. A California kneading compactor was used for specimen preparation. For the testing of treated materials, the method recommended the use of Hveem equipment and modifications of test methods used by the California Division of Highways. The procedure required a total elapsed time of eight days for the evaluation of a soil sample, approximately 4.5 man-days. Details of this original procedure can be found in Mobil's Technical Report No. 714 (24).

Since field correlation data was not available in the early stages of the research, the criteria required for evaluation of test results were not possible. It was suggested to evaluate all the obtained data for each sample individually and to base recommendations on these and associated tests that might be conducted by the customer concerned (24). After an extensive study to evaluate the test methods (15), it was found that alternative test procedures such as Marshall or California bearing ratio (CBR) gave similar results in terms of mix design, when a curing period was included prior to testing.

In 1976, Bowering and Martin (6) developed a table relating soil type, as classified by the Unified method, to the required foamed asphalt content. This table was based on the results of 50 materials examined by Mobil Oil of Australia. The full, as well as optimum, range (best mix) of asphalt content were given for each soil group. Also, the suitability of each group for stabilization with foamed asphalt was classified as good, fair or poor (see Table I).

Conoco Inc. Mix Design Procedure (12)

This procedure was divided into two sections. The first section covered the preparation of foamed asphalt test mixes simulating field operations up to compaction on a project. The second section proposed a scheme of procedures for specimen curing and moisture exposure applicable to foamed asphalt mixes. It provided specimens representative of three stages in the field cure process yet required less laboratory preparation and specimen time in molds. In the first stage called short curing, freshly compacted specimens were placed on their sides while in the mold, for 24 hours at room temperature. Intermediate curing was accomplished by removing the short cure specimens from the mold and drying them in a 104^{o}F (40^{o}C) oven for 24 hours. Drying was extended to 72 hours under the same conditions for long term curing.

The testing phase of the procedure did not recommend specific test methods. The established criteria for emulsified asphalt mix-design were recommended together with some adjustments to judge test results for foamed asphalt mixes. In addition to the table of representative data (Table I), developed by Mobil Oil, a table relating the estimated

TABLE I

REPRESENTATIVE DATA FOR FOAM TREATED MATERIALS (AFTER BOWERING AND MARTIN, 6)

Soil Group (Unified-Soil) Classification	Suitability for Use With Foam	Range of Asphalt Cement		Cohesion	Gravel Equivalency of Mix	Remarks
		Full Range	Optimum Range			
GW	Good	1.5 - 5.0	2.0 - 2.5	300 - 700	1.25 - 1.5	Permeable Mixture
GW-GC						
GW-GM	Good	1.5 - 5.5	2.0 - 4.5	300 - 400	1.25 - 1.33	Permeable Mixture
GP-GC	Good	1.5 - 4.0	2.5 - 3.0	300 - 400	1.25 - 1.33	Low Permeability
GC	Poor	4.0 - 6.0	4.0 - 6.0	300 - 400	Nil	Impermeable Bitumen Content Critical
SW	Fair	3.5 - 5.0	4.0 - 5.0	100	1.0 - 1.33	Needs Addition of -200 Filler
SW-SM	Good	1.0 - 6.0	2.5 - 4.0	100 - 400	Nil	
SP-SM	Poor	4.5 - 6.0	3.0 - 4.5	100	1.0 - 1.25	Needs Low Pen. Asphalt
SP	Fair	1.0 - 6.0	2.5 - 5.0	100 - 300	1.0 - 1.33	May Need Addition of Filler
SM	Good	1.5 - 6.0	2.5 - 4.5	100 - 400	1.33 - 1.5	
SM-SC	Good	2.5 - 6.0	4.0	400 - 700	1.33 - 1.5	
SC	Alone-Poor (+Lime-Good)	3.5 - 6.0	4.0 - 6.0 3.0 - 4.0	400 - 700	1.33 - 1.5	Needs Addition of Lime

center of foamed asphalt content range to the soil gradation properties has been developed (Table II).

The Indiana Mix Design Procedure (19)

This procedure covered the selection, proportioning and testing of components of foamed asphalt paving mixtures. It followed procedures developed before (6, 12, 16) with some modifications contributed from the experience in the state of Indiana in the field of foamed mixes. No significant changes from the procedure proposed by Conoco could be detected.

Field Construction and Performance

Numerous projects have been constructed,using foamed asphalt mixes, in the United States and in other countries (5). This section will review some of these projects as to construction methods and performance.

In 1962, three major foamed asphalt projects (26) were constructed in Arizona to connect Tuba City and Shiprock on U.S. 160. Blown sand, 100 percent passing the No. 40 sieve, was stabilized with foamed asphalt to construct a 3 inch base course overlying a 3 inch sand-emulsion subbase. In the first project, AU-1[24], the mix was placed in two 1.5 inch courses. The mixing plant used to construct the project consisted of a continuous mix travel plant set up as a central plant. Asphalt cement was foamed by the original process developed, by Csanyi. The mix was placed with a laydown machine and was followed by breakdown rolling with a 10-ton tandem steel roller. The breakdown roller followed immediately behind the laydown machine and a pneumatic roller was used to complete compaction.

TABLE II

ESTIMATED CENTER OF ASPHALT RANGE FOR SOILS
(AFTER RUCKEL, ACOTT AND BOWERING, 12)

Aggregate Wet Gradation Analysis		Foamed Asphalt Cement
Pass No. 4 US Screen (Minus 4.75 mm), % Mass	Pass No. 200 US Screen (Minus 0.075 mm), % Mass	% Mass on Dry Aggregate
Less Than 50	3.0 - 5.0	3.0
	5.0 - 7.5	3.5
	7.5 - 10.0	4.0
	Above 10.0	4.5
More Than 50	3.0 - 5.0	3.5
	5.0 - 7.5	4.0
	7.5 - 10.0	4.5
	Above 10.0	5.0

On the second project, AU-1[26], the bottom course on a trial section was laid to grade with a motor patrol. On another section the material was laid in one 3 inch, instead of two 1.5 inch courses, with a laydown machine without apparent loss of smoothness. On project AU-1[27], the third project, a pugmill of the type designed for high production of soil cement mixes fitted with asphalt foaming equipment was used.

From construction experience gained on the three projects, it was concluded that better workability and greater stability could be achieved by using a motor patrol (26). The three Arizona projects were inspected in 1980 after approximately eighteen years under traffic (27). It was observed that both shrinkage and alligator cracks were present but with little or no spalling in evidence. The cracking varied in degree over the three projects with direct relation to the type of subgrade.

In Australia, two projects using foamed asphalt were constructed during the early 70s (6). On the first project, Brown's Road, an 8 inch base course was constructed by stabilizing the widely available dune sand with 5 percent foamed asphalt by dry weight of sand. Initial compaction was achieved using a seven tire pneumatic roller. Compaction was continued using a 9-ton, 11-wheel pneumatic roller to reach 94 to 97 percent of maximum density of the mix. The other project, Springvale Road, involved upgrading crushed basalt using foamed asphalt. The foamed asphalt black base material was mixed in a stationary continuous twin-shafted pugmill with square paddle tips. The mix was either paver or grader spread to construct a 6 inch base.

No mechanical or workability problems were experienced during the construction of the two Australian projects. Unexpected rainfalls at the site of Brown's Road resulted in some curing problems. However, a three fold increase in the dynamic modulus was gained after four months. Field measurements on the Springvale Road proved that stabilizing basalt with 4 percent foamed asphalt by dry weight of basalt provided a black base as attractive as conventional hot mix asphalt with an equivalency ratio of 1.5.

A new in situ construction method was developed in Michigan in 1981 (18). They used a single pass stabilizer which had four shafts that made up the cutting and mixing chambers. The first shaft is called the cutting shaft, in that it cuts the grade to a predetermined depth. The second shaft trims the grade smoothly and places the material into the mixing portion of the machine. The last two shafts mix the material from the grade with the foamed asphalt as it is delivered from the spray bars. These machines are available in widths of 10 feet, 8 feet and 5 feet.

Descriptions of the foamed asphalt projects can be found in the literature (28, 29, 30). In general, foamed asphalt pavements can be built both by standard construction equipment and slightly modified mix plants. Some experience is needed in the construction of the relatively new mixture. The lack of field experience and/or a predefined mix design procedure (26, 27, 28, 30) were the main reasons cited for construction problems and/or poor performance of the finished pavements.

Thickness Design of Foamed Asphalt Pavements

The efficient design of pavements requires that the relative performance or relative thickness coefficient (related to high quality conventional asphalt concrete) of different materials be known. The thickness design may be determined by full scale field testing, theoretical calculation from elastic moduli and Poisson's ratio, or, from correlation of specific test properties with actual performance.

Bowering and Martin (6) found that 4 percent foamed asphalt treated crushed rock mixtures could have gravel equivalents of 1.15 to 1.4. That is, 1.15 to 1.4 inches of untreated aggregate are equivalent to 1.0 inch of treated material. These findings were based on Zube's proposal (31) that the cohesion value of asphalt treated material could be used to obtain a measure of its gravel equivalency. In the same study, an application of multilayer elastic theory using a computer program indicated a "relative thickness coefficient" in the range of 1.5 to 1.6 for a 4 percent foam treated Sydney breccia. That is, 1.5 to 1.6 inches of treated aggregate are equivalent to 1 inch of asphalt concrete.

In a joint research project between Ohio State University and Mobil Research and Development Corporation, Majidzadeh (32) proposed the thickness equivalencies of three different foamed asphalt mixes. These thickness equivalencies were based on 1.0 inch of standard asphalt mixture as the reference material. He found that all three foamed mixes had thickness equivalencies lower than 1.0. To simulate the structural response of pavement systems, he used a multilayer elastic stress distribution. Thickness equivalency that was proposed was limited only to simulation of non-failure responses of the pavement structure, such as stresses and strains in the critical regions of the structure. A flow diagram of the method is shown in Figure 2. All pavement materials were classified by their elastic constants M_R and μ. Two hypothetical pavement structures were selected for mathematical simulation. The first, Figure 3(a), was a two-layer elastic system with an asphaltic pavement resting on an elastic subgrade. The pavement performance was evaluated using the

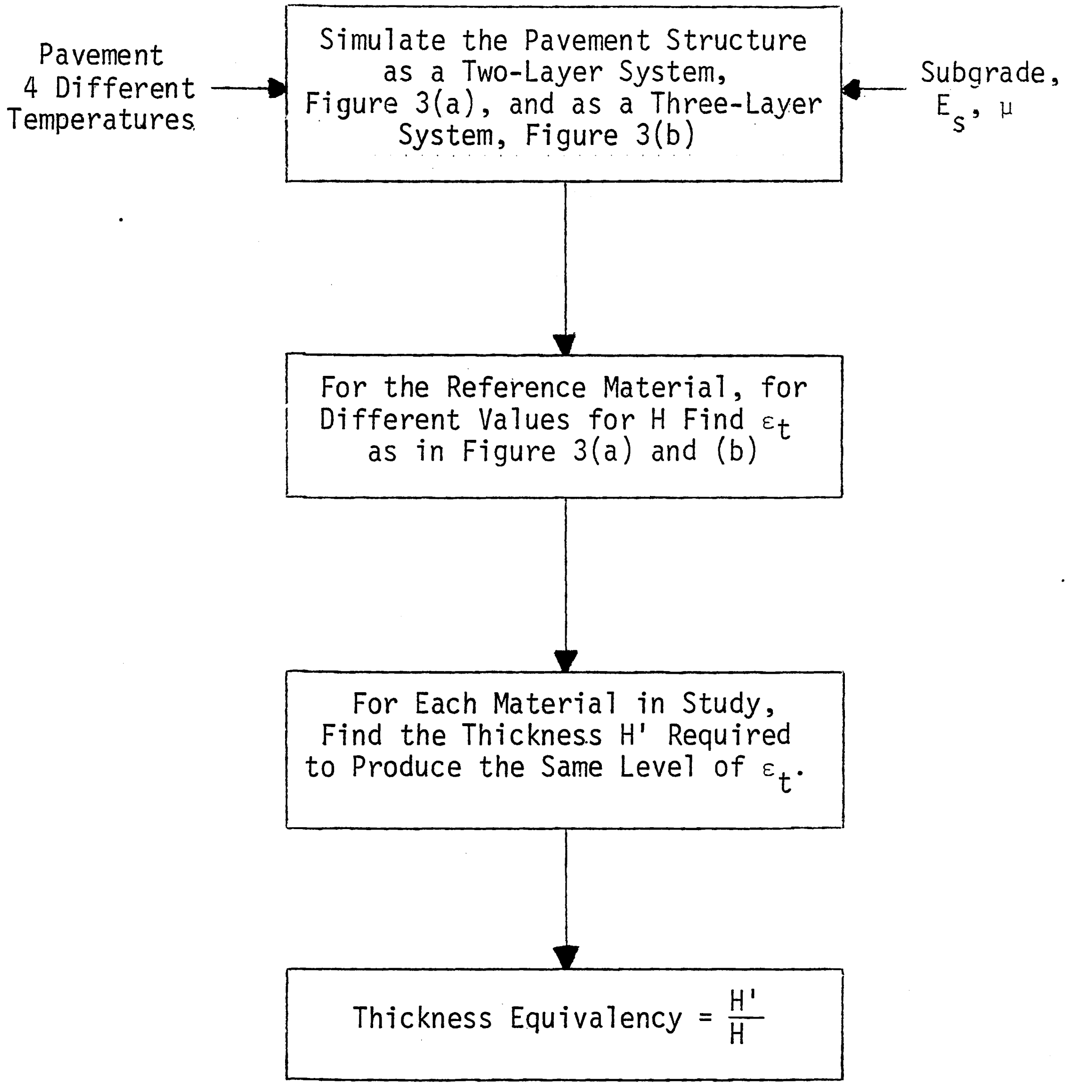


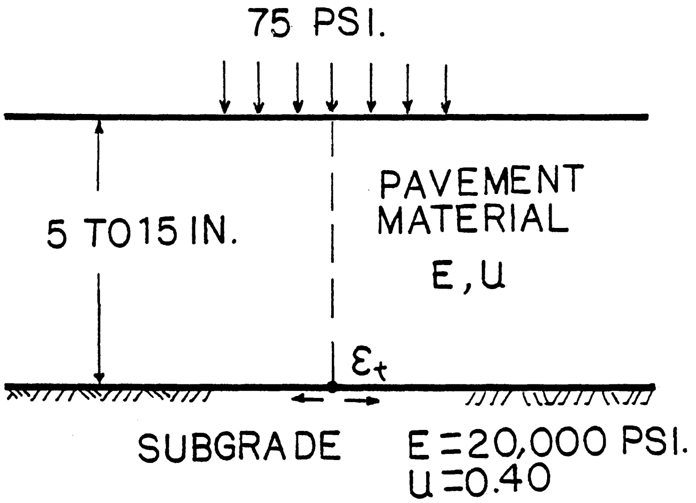
Figure 2. Scheme Used to Develop Thickness Equivalency Ratios Based on ε_t

tensile strain, ε_t, at the bottom of the asphalt layer. Pavement thickness (H) was varied between 5 and 15 inches, and the resulting tensile strains, at various temperatures were computed. For each of the materials in the study and the reference material the thickness required to produce an equal tensile strain, ε_t, level were computed. Considering that at equal levels of tensile strain the pavement structure would exhibit an equal level of performance, the structural layer equivalency could be determined as the ratio of thicknesses corresponding to equal tensile strain levels.

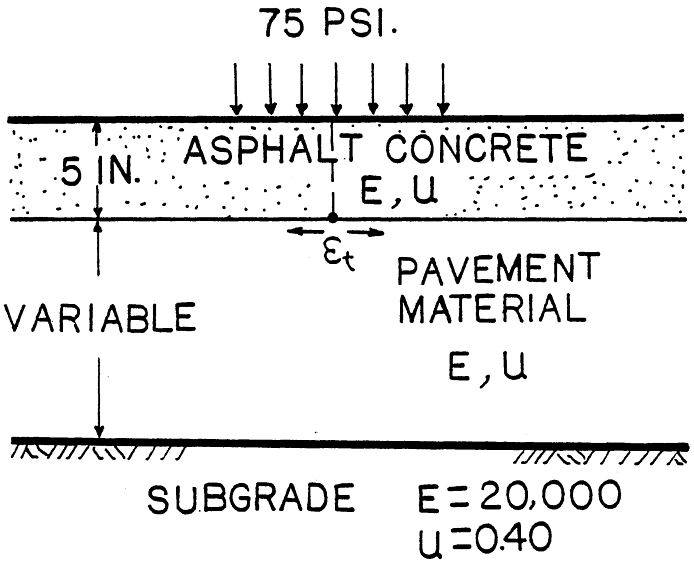
In a similar mathematical simulation, the pavement structure was assumed to be constructed with a three layer system, Figure 3(b). The paving mixtures in the study were selected as a base course, overlaid by a 5 inch asphaltic concrete surface. The tensile strain, ε_t, at the bottom of the surface course was selected as the performance criterion. Similar to the procedure used for the two-layer system the layer equivalencies were determined by computing the thicknesses (H') required to produce an equal level of tensile strain.

Results of this analysis indicated that the thickness layer equivalencies were dependent upon the pavement layer arrangements and the structural design of the pavement. It was noted that the positioning of these mixtures as a base for an asphaltic concrete surface course significantly alters the relative structural values of the four mixtures. Thickness equivalencies for the two different arrangements of the pavement structure are shown in Table III.

Little and Epps (9) have conducted a very useful study to evaluate the potential of some Texas foamed asphalt mixtures as base courses. Three criteria were taken into consideration. The first was the distri-



(a) Two Layer System



(b) Three Layer System

Figure 3. Pavements Structure Systems (After Majidzadeh, 32)

TABLE III

THICKNESS EQUIVALENCIES AT DIFFERENT PAVEMENT TEMPERATURES (AFTER MAJIDZADEH, 32)

Mix	Temperature			
	34°F	77°F	95°F	120°F
(a) Thickness Equivalencies Based on Two-Layer System				
E-322	1.00	1.00	1.00	1.00
E-320	.57	.50	.46	.46
F-322	.63	.56	.54	.52
F-328	.83	.85	.87	.80
(b) Thickness Equivalencies Based on Three-Layer System				
E-322	1.00	1.00	1.00	1.00
F-320	.72	.75	.76	.83
F-322	.5	.46	.60	.78
F-328	.95	.98	1.0	1.06

bution of vertical stress. The greater the ratio of the elastic modulus of the reinforcing layer to that of the supporting layer, the greater the success in distributing the vertical stress. As a consequence of the increase in the reinforcing action, the shear stresses in the reinforcing layer build up and become critical. Thus, the second criterion was to design the pavement to resist the shearing failure. The third criterion which they considered was the fatigue life of the pavement as measured by the maximum tensile strain in the bottom of the foamed asphalt layer.

To evaluate the ability to distribute vertical strain and thus reduce vertical subgrade deflection, foamed asphalt stabilized bases were compared to high quality asphalt base materials used on loop 4 at the AASHTO Road Test. The comparison was made by two methods. First, AASHTO structural layer coefficients of the foamed asphalt were calculated and compared to those established for bituminous stabilized bases at the AASHTO Road Test. The AASHTO materials were characterized elastically based on the work of Finn et al. (33). The methodology used to develop the structural layer a_2 coefficients was based on the vertical subgrade deformation as the governing criterion (34). Second, equivalent thicknesses were evaluated between the foamed asphalt and the AASHTO high quality asphalt concrete. The computation of thickness equivalencies was based on the maximum vertical strain, ε_v, under a dual 4500 lb wheel load. The pavement structure system which was used in the analysis was similar to that of Figure 3(a).

The authors considered the thickness equivalency scheme to be more rational than the structural layer coefficient scheme. They also pointed out that the developed a_2's should only be used for comparison purposes and not for design. Values of a_2's and thickness equivalencies are shown in Tables IV and V, respectively.

Thickness equivalencies based on the fatigue life characteristics of some Texas foamed asphalt mixes were also developed by Epps (9). Controlled stress beam fatigue tests were performed using three foamed asphalt mixtures to develop laboratory fatigue curves relating the tensile strain, ε_t, with the number of repetition to failure N_f. The general procedure was to compare the respective foamed asphalt mixtures to the AASHO asphalt bound materials based on their fatigue properties. Equivalency ratios of 2 to 4 found during that study were considered very high (9).

TABLE IV

STRUCTURAL LAYER COEFFICIENTS (a_2) COMPUTED FOR FOAMED MIXES (AFTER LITTLE, BUTTON AND EPPS, 9)

Mixture Identification	Weighted Annual Pavement Temp. °F	Structural Layer Coefficient for Base Courses Thicknesses in Inches				a_2 Avg.
		4	8	12	18	
Foamix-1	68	.34	.29	.27	.24	.28
	82	.26	.22	.20	.18	.21
Foamix-2	68	.35	.31	.29	.25	.30
	82	.27	.23	.21	.19	.22
Foamix-3	68	.29	.25	.23	.21	.24
	82	.21	.17	.15	.14	.17
Foamix-4	68	.42	.37	.34	.29	.35
	82	.32	.28	.26	.22	.27
Foamix-5	68	.30	.26	.24	.22	.25
	82	.22	.18	.17	.15	.18
High Quality Bitumen Stabilized Base (AASHTO Road Test)	68	.44	.39	.35	.31	.37
	82	.39	.34	.30	.26	.32

TABLE V

THICKNESS EQUIVALENCIES BASED ON VERTICAL SUBGRADE STRAIN CRITERION AND RELATED TO AASHTO HMAC (AFTER LITTLE, BUTTON AND EPPS, 9)

Mixture Identification	Weighted Annual Pavement Temp. °F	Subgrade Modulus x 10^3, psi	Thickness Equivalency
Foamix-1	68	3	1.65
		30	1.55
	82	3	1.67
		30	1.51
Foamix-2	68	3	1.53
		30	1.50
	82	3	1.60
		30	1.46
Foamix-3	68	3	1.81
		30	1.67
	82	3	2.14
		30	1.72
Foamix-4	68	3	1.18
		30	1.16
	82	3	1.29
		30	1.27
Foamix-5	68	3	2.00
		30	1.81
	82	3	2.16
		30	1.78

CHAPTER III

ASPHALT CEMENT AND AGGREGATE MATERIALS

Asphalt Cement

It was decided to limit this investigation of foamed asphalt mixes to the use of an 85-100 penetration paving grade asphalt cement. This grade of asphalt cement is commonly used in asphalt pavement construction in Oklahoma and is specifically required by the Oklahoma Department of Transportation in all plant mix bituminous bases and surface course mixes (35). Also a recent investigation found no appreciable differences in the stabilities of foamed asphalt mixes made with different grades of asphalt cement (10), so it appeared that this was a valid decision and should not restrict the applicability of the research results.

A twenty gallon sample of the 85-100 penetration asphalt cement was obtained from the Honegger Construction Company's "Foamix" plant site near Yukon, Oklahoma. At the time the sample was acquired, the company's plant was producing a fine aggregate-foamed asphalt base course material that was being placed on a nearby county road. The asphalt cement was purchased from the Allied Materials Corporation refinery at Stroud, Oklahoma.

Laboratory Tests

After obtaining a supply of the asphalt cement from the Honegger plant, samples were subjected to routine laboratory tests to verify its grade classification and other pertinent characteristics. The results of these standard laboratory tests are given in Table VI.

Aggregates

Since little attempt has been made in Oklahoma to study or take advantage of foamed asphalt mixing for road construction purposes, one of the major incentives for this work was a desire to demonstrate the efficacy and applicability of this mixing procedure through the evaluation of a number of fine aggregate-foamed asphalt mixtures formulated from materials indigenous to Oklahoma.

Both wind and water borne deposits of fine aggregate materials are readily available throughout most areas of the state. However, certain physical characteristics of fine aggregates, such as gradation, particle angularity and surface texture are thought to affect the suitability of the aggregate for good mixing with foamed asphalt and influence the stability and other properties of compacted mixtures. The aggregates selected should encompass a range of these properties in order to determine their significance.

This presented some difficulty, however, as tests on a large number of samples from a variety of sources in central Oklahoma indicated little difference in gradation and or angularity of particles. Ultimately, it was decided to select a naturally occurring sand, a processed sand from a screening and washing operation, and a fine aggregate material from a crushing operation. Two additional fine aggregates were then manufactured by blending and crushing these basic materials to obtain the desired range of properties.

TABLE VI

CHARACTERISTICS OF ASPHALT CEMENT

Characteristics	ASTM[1] Method of Test	Test Value
Penetration at 25°C (77°F) 100g, 5 sec., 0.10 mm	D5	91
Ductility at 25°C (77°F) 5 cm. per min., cm.	D113	150+
Viscosity at 135°C (275°F), cSt	D2170	130
Thin-film oven test, 1/8 in (3.2 mm), 163°C (325°F), 5 hour	D1754	
Loss on heating, percent		0.24
Penetration, of residue, percent of original		68
Ductility of residue at 25°C (77°F), 5 cm. per min., cm		101
Specific gravity at 25°C (77°F)	D70	.994
Softening point, ring and ball	D36	110°F
Flash point, COC	D92	650°F

[1] 1983 Annual Book of ASTM Standards, Section 4, Volumes 04.03 and 04.04.

Thus, five fine aggregate materials were used in the various aspects of this study. These materials were:

1. Sand (N-1). A very fine, generally silicious, silty sand from alluvium deposits along the North Canadian River in Canadian County, Oklahoma. This sand was obtained at a foamix plant site northwest of Yukon, Oklahoma. Note: the bracketed alpha-numeric characters were used to identify each of the respective fine aggregate materials.

2. Washed Sand (S-1). A clean relatively coarse reddish-brown sand produced from screening and washing operation at a gravel pit northeast of Asher, Oklahoma. This pit is located in the Maud conglomerate of the Wellington-Admire Geologic Unit (36) in Pottawatomie County. The Conglomerate deposits consist of a fairly homogenous mixture of chert particles ranging in gradation from fine sand to cobbles. Foamed mixes made from this sand exhibited poor workability due primarily to the gradation of the material. Because of this, only one series of tests were conducted on mixes made exclusively from this sand.

3. Screenings (C-1). Crushed limestone screenings from a quarry and crushing operation northeast of Drumright, Oklahoma. The quarry is located in the Lecompton Geologic Unit (36) in Creek County. This unit consists of gray thin-bedded to massive limestone that is hard and dense. At the quarry site the limestone bed has a thickness of about 12 feet and an overburden of 10 to 18 feet of shale and sandstone. The individual particles of this material had a relatively high degree of angularity and a rough surface texture.

4. Blended Sand (B-1). A mixture of fine sand (N-1) and washed sand (S-1) blended in a one to one ratio by weight. The two fold objective of using this blended fine aggregate was to 1) see if the addition

of fines would improve the workability and the properties of the coarse sand foamixes, and, at the same time, 2) whether the addition of coarser material would enhance the properties of the foamixes made from the finer sand.

5. Crushed Sand (C-2). Pit run chert gravel from the Asher, Oklahoma, pit was crushed in a small jaw crusher. The material was then passed through a 3/8 in. sieve to remove the coarser particles. This produced a fine aggregate having a degree of angularity higher than the (N-1), (S-1) and (B-1) materials.

Aggregate Properties

For the most part, standard laboratory test procedures were used to determine certain physical properties of the fine aggregates. Based on the review of literature, these are the aggregate properties that are considered to affect the quality of foamed asphalt mixtures. The results of both the standard and non-standard tests are presented in Table VII. The non-standard test procedures used are more fully discussed below.

1. Modified Particle Index Test: Some investigators (37, 38) have successfully used the particle index test to indicate the effects of particle angularity and surface roughness on the compaction and strength characteristics of asphalt-aggregate mixtures. The standard test procedure (ASTM D3398) suggests modifying the size of the mold and the amount of rodding effort used when testing fine sieve fractions of aggregates passing the No. 4 (4.75 mm) sieve.

For this study, the standard test and calculation procedures were used with the following modifications in test equipment.

TABLE VII

PHYSICAL PROPERTIES OF FINE AGGREGATES

Characteristic	ASTM Method of Test	Test Values (N-1)	(S-1)	(B-1)	(C-1)	(C-2)
Sieve Analysis	C136	See Figure 4			See Figure 5	
Optimum M/C	D698	11.0	11.3	10.0	9.8	10.1
Specific Gravity	C128	2.43	2.41	2.42	2.6	2.41
Liquid Limit	D423	NP	NP	NP	NP	NP
Plasticity Index	D424	NP	NP	NP	NP	NP
Particle Index	D3398*	5.3	6.3	5.8	15.5	12.9
Fluff Point	**	7.5	7.6	4.0	4.6	5.5

*Modified Test Procedure.

**Non-Standard Test Procedure.

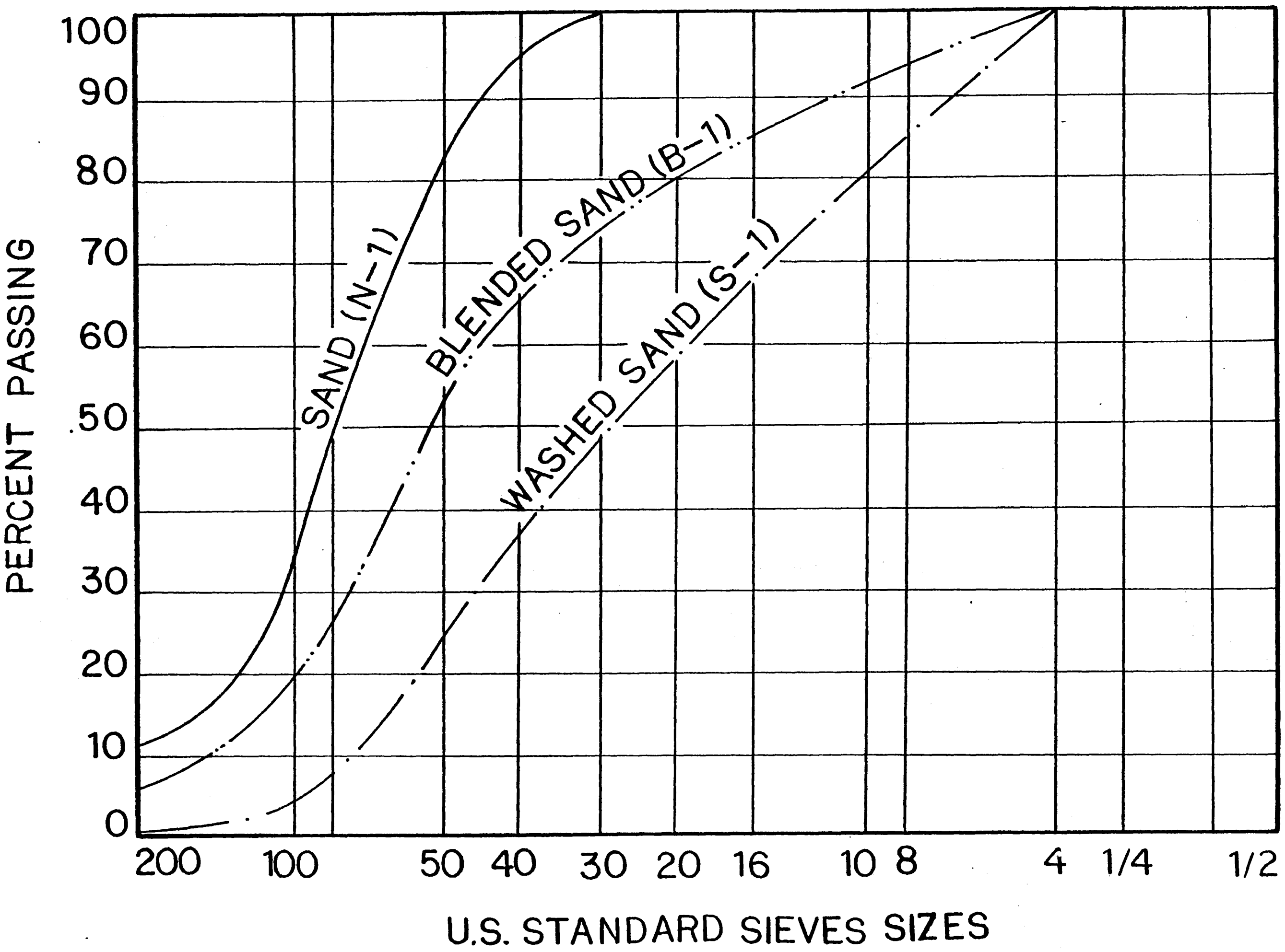


Figure 4. Gradation of Aggregates (N-1), (S-1) and (B-1)

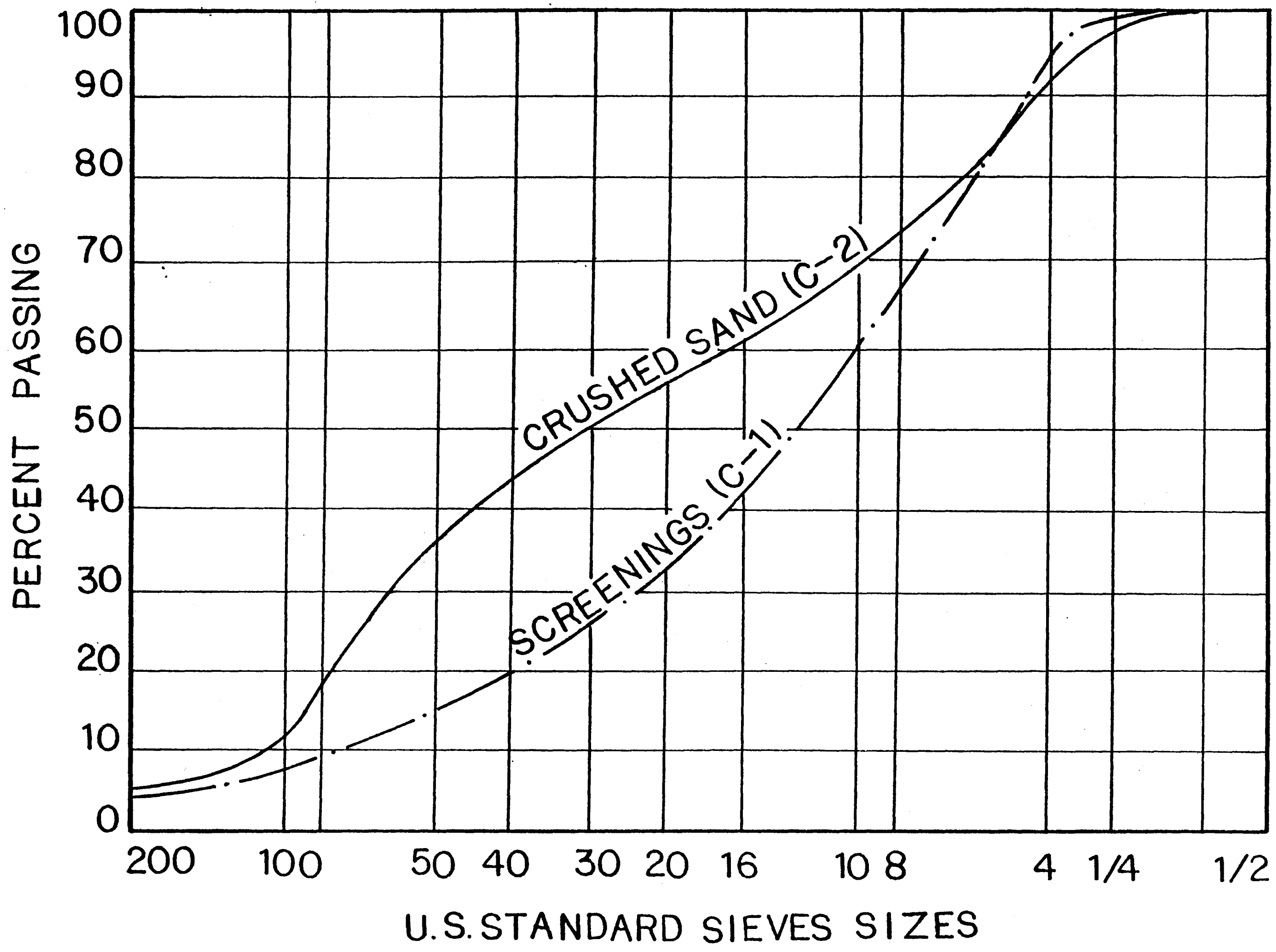


Figure 5. Gradation of Aggregates (C-1) and (C-2)

a. Cylindrical Mold: A mold having an inside diameter of 3.0 in (76.2 mm) and a height of 3.5 in (88.9 mm) was used. These dimensions are exactly one half of those specified in the standard test and reduced the volume of material required for each sieve fraction by approximately 88 percent.

b. Tamping Rod: A steel tamping rod with a diameter of 0.312 in (7.94 mm), a length of 12.0 in (305.0 mm), and a weight of about 0.25 lbs (115.0 g) was used. The tamping end of this rod was rounded to a hemispherical tip. These dimensions of the tamping rod are exactly one half of those specified and the weight was reduced to about one eighth of that used in the standard test.

c. Height Control Sleeve: A steel tube having an inside diameter of 0.375 in (9.5 mm) and a 10 in (254.0 mm) length was used to control the height of drop of the tamping rod. The arrangement was similar to that used on a standard Proctor rammer with the rod pinned in the sleeve so that it could be dropped vertically exactly 2.0 in (5.1 mm) when the bottom of the sleeve was in contact with the surface of the aggregate.

This equipment is shown in Figure 6 and its use resulted in approximately the same amount of rodding effort per cubic inch of aggregate, i.e., 0.61 in-lb/in^3, as in the standard test.

2. Fluff Point Test: The purpose of this non-standard test is to find the aggregate moisture content which is required to bring it to its maximum loose volume. The fluff point moisture content has been used by a number of investigators (19, 23) as the mixing and molding moisture content of foamed asphalt half-fine aggregate mixtures. However, Epps (9) found that additional wetting of the aggregates he used appeared to improve dispersion of the foamed asphalt during mixing. In

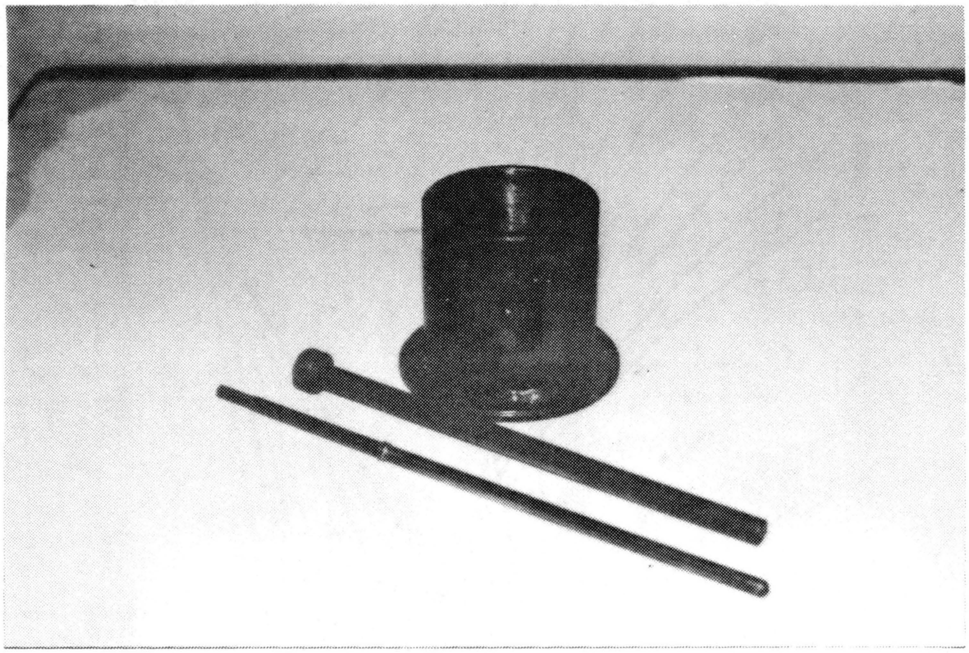


Figure 6. Particle Index Mold and Tamping Rod

this study the fluff points of the aggregates were determined to see if this moisture content (fluff point) would correlate with their optimum premolding moisture contents.

The "fluff point" of an aggregate is determined by mixing a sample of the dry material with increasing amounts of added water and measuring the wet loose volume of the aggregate sample at each increment of added moisture content. The moisture content at which the maximum bulk volume of the aggregate is obtained is considered the "fluff point" of the material. The equipment used for this test is shown in Figure 7. The procedures used to determine the fluff point moisture contents for the five fine aggregates was as follows:

1. A batch weighing 500.0 g of the dry aggregate was mixed thoroughly, in a mixing bowl, with 1.0 percent water by dry weight of aggregate (5.0 g).

2. The wet aggregate was poured in a graduated cylinder (capable of measuring up to 500.0 ml to the nearest 5.0 ml), the cylinder was lightly shaken, and the loose volume of the wet material was measured.

3. The wet aggregate was then poured from the graduated cylinder back to the mixing bowl and the moisture content was raised an increment of 1.0 percent by dry weight of aggregate (in our case it was 5.0 g).

4. Steps 2 and 3 were repeated until the volume of moist aggregate, as measured by the graduated cylinder, started to decrease.

5. The data from previous steps permitted the development of a curve relating the moisture content and the loose volume of the wet aggregate.

6. The "Fluff Point" was determined as the aggregate moisture content at which the maximum loose volume of the moist aggregate was attained.

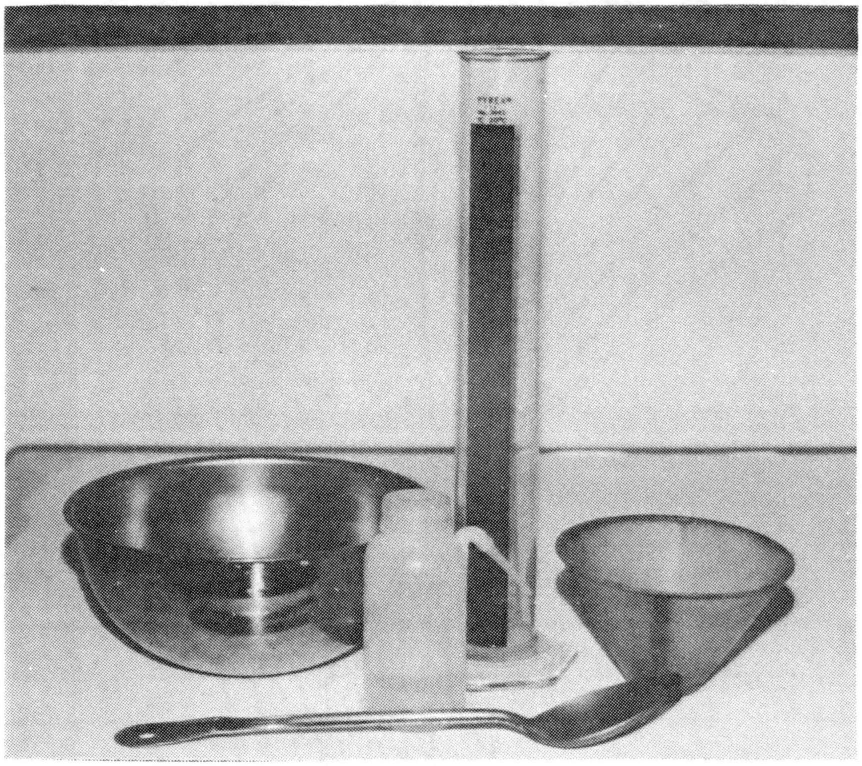


Figure 7. Fluff Point Test Apparatus

CHAPTER IV

LABORATORY EQUIPMENT AND PROCEDURES

Mixing and Molding Equipment

Foamed Asphalt Dispenser

A laboratory foamix asphalt dispenser for use in the study was obtained through a lease purchase agreement with Conoco, Inc. This dispenser unit was built in the Technical Services Laboratory of Conoco's facilities in Ponca City, Oklahoma, and was designed to simulate foamed asphalt produced in commercial mixing units. The unit weighs approximately 300 lbs (136 kgs) and is 42 in (1070 mm) long, 18 in (460 mm) wide and 41 in (1041 mm) high from the bottom of the baseplate mount to the asphalt tank lid. A photograph of the dispenser is shown in Figure 8.

Asphalt is electrically heated in a thermostatically controlled 2-gallon (7.75-liter) tank. An asphalt pump at the bottom of the tank circulates hot asphalt through the electrically heated and well insulated piping network. The panel on the right has controls for automatic operation of the system, i.e., main power and pump switches, heating system controller, and timers and regulators for metering the flow of air, water and hot asphalt through the specially designed foam nozzle, located at the left.

The operational procedure for the foam dispenser is detailed in Appendix B and can be summarized as follows:

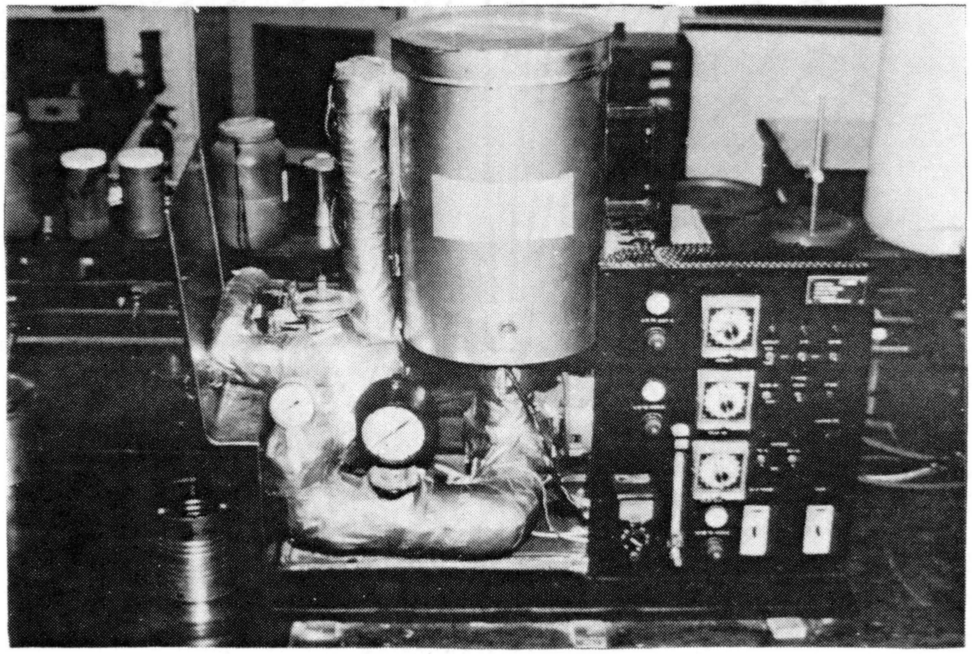


Figure 8. Laboratory Foamed Asphalt Dispenser Obtained From Conoco, Inc.

1. Asphalt contained in the storage tank and the piping network is preheated to at least 275^{o}F (135^{o}C) before starting the circulating gear pump. This requires approximately 3 hours as the asphalt lines, i.e., the pipe network must be heated gradually.

2. Hot asphalt from the tank is then circulated by the pump through the system at a constant rate of 0.50 gallons per minute.

3. Water at ambient temperature is atomized by compressed air and flows to a mixing chamber in the foam nozzle where it is mixed with hot asphalt cement. The required amount of air and water for optimum foam quality is regulated by a flowmeter. The desired flowmeter setting is obtained from a calibration curve, Figure 9, which shows the setting versus water flow as a weight percent of the asphalt flow.

4. When the addition of foamed asphalt to a test batch of aggregate is desired, the asphalt flow is switched from circulate to the foam nozzle (through a 3-way air actuated valve) for a selected time interval to deliver the desired weight of asphalt for the batch. Figure 10 is the calibration chart used to determine the time setting required for delivery of a prescribed weight of asphalt. Sample calculations using Figures 9 and 10 for setting the air, water and asphalt timers are presented in Appendix B.

Two major problems were encountered in the operation of the foam dispenser. The first problem involved repeated failure of the two 115-volt AC electric heating tapes used to heat the asphalt lines. Installation of new tapes required complete removal and replacement of the insulating material around the pipe network. Finally, the two heating tapes were connected in parallel rather than in series to avoid the large voltage drop between the tapes which was believed to cause the

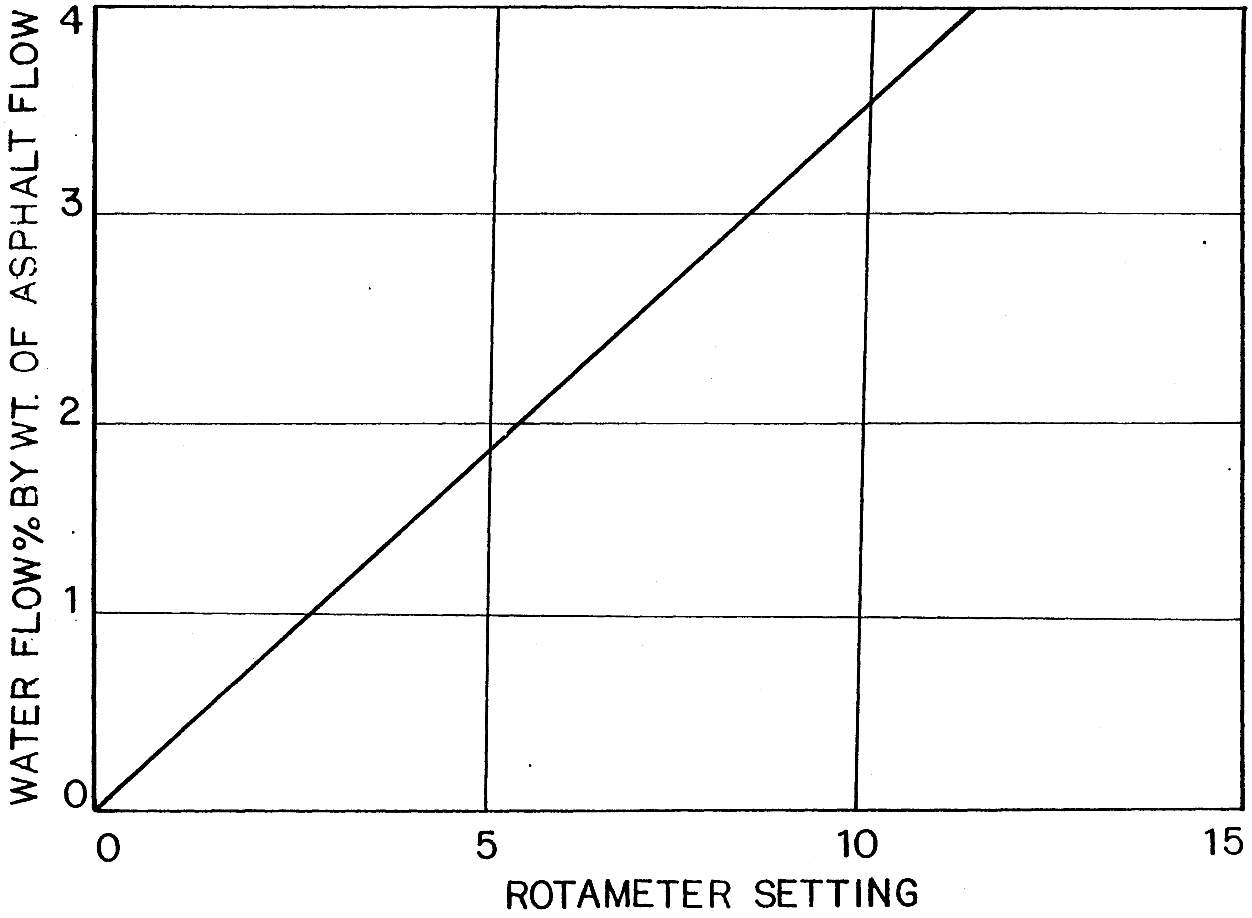


Figure 9. Water Flow Rate Vs. Rotameter Setting for Laboratory Foam Dispenser (After Conoco, Inc., 7)

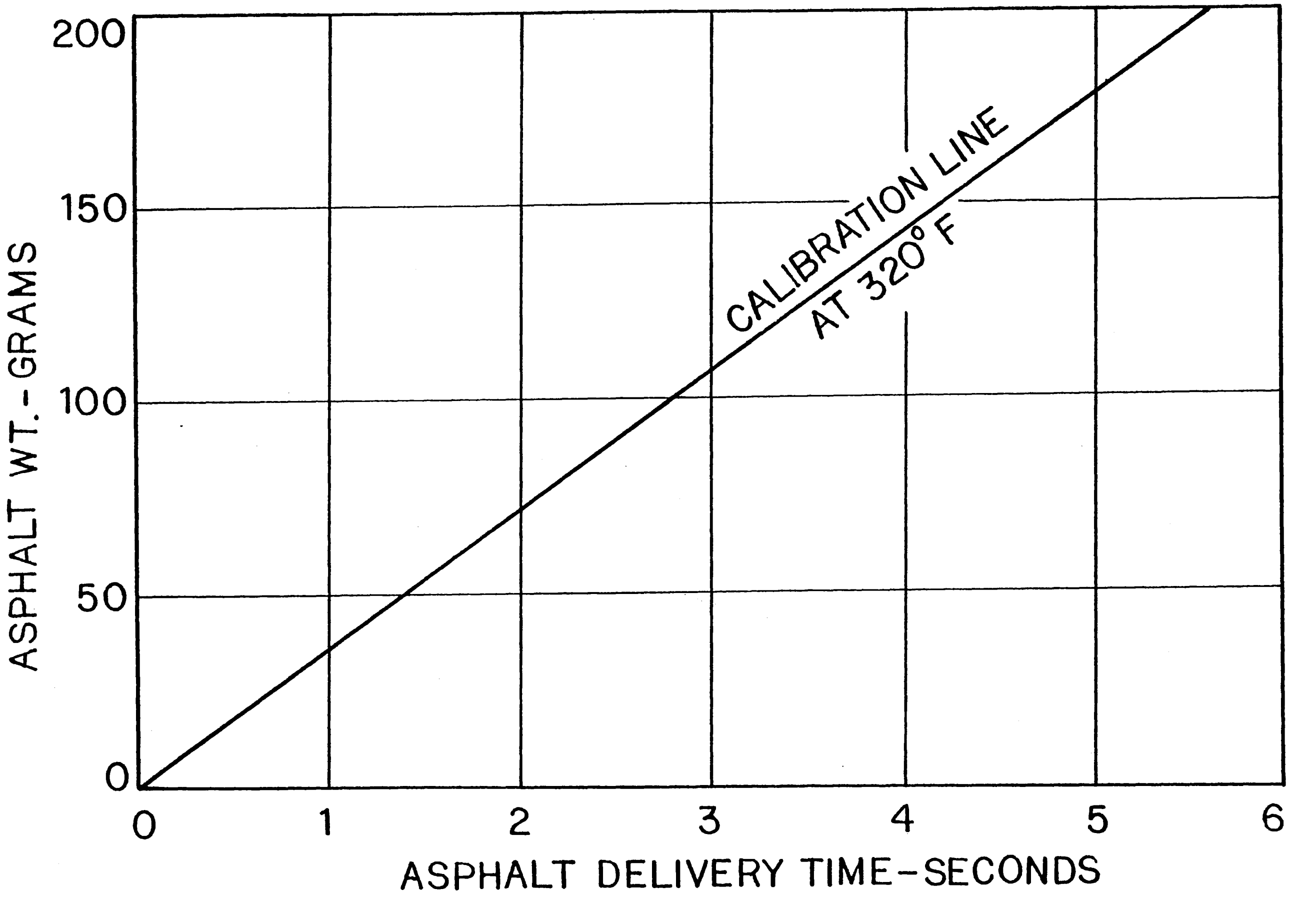


Figure 10. Asphalt Weight Vs. Delivery Time for the Laboratory Foam Dispenser (After Conoco, Inc., 7)

frequent failures. The recommended operating procedure was also modified to raise the line temperature gradually in 25^{o}F (14^{o}C) increments and allow the line temperature to stabilize after each incremental raise.

The second major problem occurred when the 1/3 HP pump motor burned out while attempting to start it before the pump and line system were up to operating temperature. Although protected by a 10 ampere circuit breaker, further protection was added to the motor by the installation of an 8 ampere dual element fuse in the motor wiring.

Mechanical Mixer

A three speed Hobart C-100 orbital food mixer equipped with a flat beater type agitator was used to blend the foamed asphalt with the various fine aggregate batches. The mixer was placed on a 15 by 17 in (380 by 420 mm) piece of 3/4 in (19 mm) plywood with 1/2 in (13 mm) steel ball bearings set in drilled holes on the bottom side. This arrangement allowed mobility of the mixer on the steel surfaced counter and facilitated positioning the mixing bowl beneath the foam nozzle of the dispenser.

Specimen Molding Equipment

The mechanical gyratory-shear molding equipment used was similar to that developed by the Texas Department of Transportation (39) and used by the Oklahoma Department of Transportation (40). This equipment conveys a gyratory-shearing action to asphalt-aggregate mixtures in a mold at low initial pressure to achieve optimal orientation of the aggregate particles. During compaction the platen of this compactor rotates and the specially designed mold imparts a rocking type of shearing action to the compressed mixture in the mold. The compacted cylindrical test specimens were 2.0 in (51 mm) in height and 4.0 in (102 mm) in diameter.

One series of test specimens of the foamed asphalt-fine aggregate mixtures were made using standard CBR molds, 6.0 in (15.3 mm) in diameter by 7.0 in (178 mm) in height, and the 10.0 lb (4.54 kg.) rammer and compaction procedure specified in ASTM-D1557 (41). These specimens were cured for various periods and then subjected to vane shear testing in the laboratory.

Mixture Preparation and Procedures

A foamed asphalt mix is a combination of wet unheated aggregate and asphalt cement in which the asphalt is applied and mixed while in the foamed condition. To obtain a reasonably well dispersed foamed asphalt mixture from a given aggregate and a given asphalt cement, several important parameters of the ingredients must be determined and used in the mixing process. First, the proper quality of the foamed asphalt as indicated by its "Foam Expansion Ratio" and "Foam Half Life" is determined (see definition of these properties in Chapter II). Based on the recommendations found in the literature (10, 12), a foam expansion ratio of 8 to 15 and a half life of at least 20 seconds is desired. Second, a specific premolding moisture content of the aggregate to be used is found. This moisture content differed considerably from one material to another and varied from 50 to 100 percent of the optimum moisture content of the aggregates, as determined from the standard compaction test, ASTM-D598 (41).

For this study, the foamed asphalt mixtures were divided into three groups with each group classified according to its use in the subsequent testing program. The first group of mixtures, or "trial mixes", were needed to establish some constant values such as the weight of asphalt and aggregate that stuck to the mixer's blade and bowl and the mix batch weights required to produce the desired number of specimens. The second group of mixtures was used for the determination of the optimum premolding moisture content for each aggregate. This determination was based on the moisture content at which maximum density of a compacted foamed asphalt mixture was obtained. The third group comprised foamed asphalt mixtures which were used for the rest of the testing program. These mixtures, i.e., the third group, were prepared based on the results obtained from the first and second group of mixtures.

Establishing Control of Foamed Asphalt Quality

In order to determine the dispenser control settings to achieve the desired "Foam Expansion Ratio" and "Foam Half Life" with the asphalt cement used, a series of preliminary tests on foam batches were carried out using various control settings. Two asphalt temperatures, 325°F (163°C) and 350°F (177°C), were used with water flow rates of 1.5, 2.0 and 2.5 percent of the added asphalt weight at each of these temperatures. One gallon size empty food cans were employed to receive and measure the volume of the foamed asphalt batches. A plentiful supply of these cans was obtained free of charge and, thus, they could be discarded after use.

A volumetrically calibrated dip-stick facilitated the measurement of the volume of a batch of foam dispensed in a can and a stop watch was used to determine the elapsed time necessary for the volume of foam to subside to one-half of its initial volume. Details of this procedure and the necessary calculations can be found in Appendix A.

The asphalt cement used in this study required the addition of 0.70 percent (by weight of asphalt) of magnesium stearate, a counter anti-foaming agent, in order to obtain the proper foaming characteristics. The Optimum Foam Ratio and Half Life was 12 and 60 seconds respectively. These foam characteristics were obtained for all subsequent batches of foamed asphalt and fine aggregate that were mixed using an asphalt temperature of 325^{o}F (163^{o}C) and a flow rate of 2.0 percent water by weight of added asphalt.

General Foamixing Procedure

Following the determination of the optimum foam quality control settings, preliminary foamixes of each fine aggregate were made to check and obtain other necessary parameters of the individual materials prior to mixing and molding the specimens used for design purposes. A general foamixing procedure used for all of the various mixtures involved was as follows:

1. The foam dispenser unit was made ready for use in accordance with the operational procedure (Appendix A).

2. The previously determined settings for asphalt temperature and water flow rate to yield optimum foam quality were set on the control panel.

3. Based on the dry weight of aggregate needed to make a given number of compacted specimen, the desired weight of asphalt was calculated and this weight was increased slightly to allow for the amount lost on the mixer blade and bowl (see section on Trial Mixtures below).

4. The foam dispenser was then programmed for the asphalt delivery time corresponding to the calculated weight of asphalt using Figure 10.

5. One batch of foamed asphalt was dispensed and wasted to insure proper functioning of the foam delivery system.

6. The bowl of the mechanical mixer containing the proper weight of pre-moistened aggregate (see section on Mixtures for Molding Moisture Content) was then positioned under the dispenser nozzle, a batch of foamed asphalt added, and mixing started.

7. The mixing process was continued for 3 minutes or until the asphalt was well dispersed and the aggregate mixture noticeably darkened in appearance.

8. The mixer bowl was removed and the asphalt-aggregate mixture divided into portions necessary for molding individual specimens.

9. These portions were placed in separate covered pans when molding was done immediately following the mixing process. When molding was delayed the portions were sealed in plastic bags.

Trial Mixtures: The purpose of the trial mixtures was to determine the dry batch weights of each aggregate necessary to produce a desired number of molded specimens of the proper diameter and height when mixed with various foamed asphalt contents. Initially three 3000 g dry batches of each aggregate were prepared and enough water added to adjust the moisture to approximately 80 percent of the respective aggregate's optimum moisture content. Each of these three wet aggregate batches was mixed with a specific amount of foamed asphalt. The three levels of asphalt content used were 3.5, 5.0, and 6.5 percent by dry weight of aggregate. After molding specimens from each of the batches, they were placed in an oven and dried to constant weight. This data permitted the calculation of the necessary dry batch weights of the respective aggregates to be used in subsequent mixtures.

Prior to mixing each batch of a given aggregate, the mixing bowl and agitator blade were tared. When mixing was concluded, the mixture was removed from the bowl and loose mix on the blade was returned to the mixed batch by light brushing. The blade and bowl were then weighed to determine the amount of aggregate and asphalt clinging to their surfaces. The percent of asphalt retained or stuck to the mixer blade and bowl at each level of asphalt content was calculated and used to adjust the amount of added asphalt for a desired content level in subsequent mixtures. A detailed example of these preliminary procedures and calculations is presented in Appendix B.

<u>Mixtures for Determining Premolding Moisture Content:</u> The purpose of this group of mixtures was to find the optimum premolding moisture content of each of the five aggregates for various levels of foamed asphalt content. The optimum premolding aggregate moisture content at a given asphalt content level was considered that at which the maximum compacted dry density is achieved.

Based on the information obtained from the trial mixtures, four dry batches of each aggregate (enough to mold 3 specimens per batch) were prepared. These amounts varied from 2250 g to 3000 g for the respective aggregates. The moisture contents of the four dry batches of aggregate were adjusted to 50, 65, 80 and 95 percent (respectively) of the optimum value as determined from the ASTM-D698 test procedure for each aggregate. The added water and aggregate batches were thoroughly mixed to ensure complete dispersion of the water.

Each of the wet aggregate batches was then mixed with a different level of foamed asphalt cement, ranging from 2.0 to 5.5 percent by weight of dry aggregate, and then divided into three equal portions. The three equal portions were then compacted into 3 molded specimens about 2.0 in (51.0 mm) high and 4.0 in (102.0 mm) diameter using the gyratory-shear compactor. The height of each specimen was determined in the mold. The molded specimens were then dried to constant weight in an oven. The dry density of each specimen was determined from the data and the relation between molding moisture content and dry density of foamix for each level of foamed asphalt content was plotted. From these plots, the optimum molding moisture content for each aggregate was obtained for all subsequent mixtures made at a specific foamed asphalt content.

Mixtures for Determining Design Asphalt Content: The design or optimum foamed asphalt content for four of the selected aggregates was determined by subjecting molded specimens at various levels of asphalt content to tests for their Resilient Modulus and Hveem Resistance and Stability values. The results of these tests were used to select an optimum foamed asphalt content for each aggregate and could be used to predict probable performance of the mixtures as base material for road construction.

Batches of these four aggregates, (N-1), (B-1), (C-1) and (C-2) were adjusted to their optimum molding moisture contents and then mixed with four levels of foamed asphalt content varying from 2.5 to 6.5 percent by dry weight of aggregate. Each batch of wet aggregate consisted of enough material to mold three gyratory-shear compacted specimens at a given foamed asphalt content. Thus, a set of molded specimens comprised three specimens at each of the four asphalt contents within the range used.

While the same set of specimens could be used for both the resilient modulus and Hveem tests, different testing temperatures were employed and different curing periods (see Curing Procedures) for the specimens were used prior to the tests. This necessitated the preparation of enough mixtures of each aggregate to satisfy the testing program given in Table VIII.

Mixtures to Study the Effect of Aggregate Particle Index: The purpose of these mixtures was to ascertain the effects of aggregate angularity and surface texture on the strength of foamed asphalt mixtures and to compare the strengths with those of hot asphalt mixes made using the same aggregates and asphalt cement. Molded specimens from both the foamed asphalt and hot asphalt mixes at various asphalt contents were tested for Marshall stability at two different temperatures.

Batches of the five aggregates, (N-1), (B-1), (C-1), (C-2) and (S-1), were mixed with foamed asphalt following the same procedure described in the previous section. Similar batches of the same five aggregates were heated to 275°F (135°C) then mixed with asphalt cement that had been heated to a temperature of 300°F (149°F). These hot mixed batches contained the same asphalt cement used for the foamix batches. Although the washed sand (S-1) was considered unsuitable for use with foamed asphalt because of poor workability, it was included in this series of mixtures to see if the lack of minus No. 200 sieve particles had any effect on the relation between the materials particle index and its strength or stability.

Since all the foamix specimens were tested after long curing (see Curing Procedures), only 4 sets of specimens; 2 sets of foamix and two sets of hot mix, were required for each of the five aggregates. The testing program is summarized in Table IX.

TABLE VIII

TESTING PROGRAM FOR MIXTURES TO DETERMINE THE DESIGN ASPHALT CONTENT

No. of Specimen Sets*	Asphalt Content Range Percent	Testing Temperature	Curing Condition
1	2.5 - 5.5	78°F (25°C)	Short
2	2.5 - 5.5	78°F (25°C) and 105°F (40°C)	Medium
2	2.5 - 5.5	78°F (25°C) and 105°F (40°C)	Long
1	3.5 - 6.5	78°F (25°C)	Long**

*A specimen set comprised three molded specimens of each aggregate at four different asphalt contents for a total of 48 specimens.

**This set of specimens was tested after exposure to moisture absorption.

TABLE IX

TESTING PROGRAM FOR MIXTURES TO STUDY THE EFFECT OF PARTICLE INDEX

No. of Specimen Sets*	Type of Mix	Asphalt Content Range Percent	Testing Temperature	Curing Condition
1	Foamed	2.5 - 5.5	78°F (25°C)	Long
1	Foamed	2.5 - 5.5	105°F (40°C)	Long
1	Foamed	2.5 - 5.5	140°F (60°C)**	Long
1	Hot	3.0 - 5.0	78°F (25°C)	None
1	Hot	3.0 - 5.0	105°F (40°C)	None

*A specimen set comprised three molded specimens of each aggregate at four different asphalt contents for a total of 48 specimens.

**Only foamixed specimens from aggregates (C-1) and (C-2) were tested at this temperature.

Specimen Molding Procedures

Gyratory-Shear Compaction

With the exception of one series of molded specimens, all test specimens were made using the previously described gyratory-shear compaction equipment. These compacted cylindrical specimens were nominally 2.0 in (51 mm) in height and 4.0 in (102 mm) in diameter. The specimens were molded in accordance with the specified compaction procedure (39, 40) deviating only in regard to mixture temperature at the time of molding. Since these foamixes are considered "cold" rather than "hot" mixtures, they were compacted at ambient laboratory temperature--approximately 78^{o}F (25^{o}C).

Modified AASHTO Compaction

One series of test specimens were molded in standard CBR molds using the rammer and compaction procedure stipulated in method B of the (ASTM D1557). These specimens were used for shear strength determinations (see Vane Shear Test). These tests required 10 molded specimens of each aggregate containing its design asphalt content.

Specimen Curing Procedure

The various test procedures were conducted on molded specimens after they had been subjected to various levels of curing. In this case, curing implies a reasonably quick process for reducing the molding moisture content of the specimens to simulate field drying conditions. Compacted foamixes can be expected to gain strength as the mixing moisture content is reduced due to drying or evaporation. The procedures for the three curing levels used in this study were as follows.

Short Curing

Gyratory shear compacted specimens were extruded from the metal mold into 2.5 m (63 mm) long sections of 4.05 in (103 mm) inside diameter PVC water pipe. This permitted in-mold curing of a number of compacted specimens without requiring separate metal molds for each of the specimens. The weight of these molded specimens and their "substitute" molds was determined immediately following the extrusion process. The molds were placed on their sides with the top and bottom surfaces of the specimen exposed and left to dry at ambient laboratory temperature, i.e., at 78 ± 5^oF (25 ± 3^oC). After 24 ± 0.5 hours of curing the specimens and molds were again weighed to determine the percent of moisture lost during curing (see Calculations in Appendix B). The specimens were then removed from the molds and tested or subjected to further out-of-mold curing, i.e., intermediate and long curing.

Intermediate Curing

Short cured specimens extruded from their molds were placed in a forced draft electric oven at 105^oF (40^oC) and left to dry for 24 ± 0.5 hours. After removing and cooling the specimens, they were again weighed to determine moisture loss and then tested or stored in sealed plastic bags, if testing was delayed.

Long Curing

For long curing, the specimens were oven dried at 105^oF (40^oC) for 72 ± 0.5 hours after short curing. After cooling and weighing, the specimens were tested or stored in sealed plastic bags until testing could be accomplished.

Testing Equipment and Procedures

Hveem Resistance and Stabilometer Values

Standard test equipment as described in ASTM D1560 (41) and D2844 (41) was used. This equipment comprised a Hveem stabilometer and a hydraulic compression testing machine capable of applying a load at a rate of 0.05 in (1.3 mm)/minute.

Procedure: Both stabilometer and resistance values of the gyratory-shear compacted specimens were determined from one stabilometer test. The procedure used conformed to that stipulated in the standard test for bituminous mixtures, i.e., ASTM D1560, with two exceptions. The specimen height was 2.0 ± 0.1 in (51.0 ± 3.0 mm) instead of the standard height of 2.5 ± 0.1 in (63 ± 3.0 mm) and the specimens were tested at temperatures of 78°F (25°C) and 105°F (40°C) rather than at the standard temperature of 140°F (60°C). The determined stabilometer values were corrected for specimen height using the standard correction chart. The resistance values were similarly corrected through extrapolation of the standard correction chart of ASTM D2844.

Precedent for the simultaneous determination of stabilometer and resistance values of the foamed asphalt mixtures was found in a recent study by Franco et al. (19). This study found that the displacement reading for cold mixed specimens did not change after repeated testing of the specimens and that this reading (in terms of the number of turns of the stabilometer pump handle) was primarily a function of the surface texture of the specimen.

Marshall Stability Values

The purpose of this test was mentioned in a previous section (see Mixtures to Study the Effects of Aggregate Particle Index). Testing equipment as described in ASTM D1559 (41) was used with the exceptions that the ring dynamometer was replaced by a 10,000 lb (4.45 x 10^4 N) capacity load cell and a strip chart recorder was employed to record directly the stability and flow values.

Procedure. The testing procedure detailed in the standard test, ASTM D1559, was followed. However, instead of testing all the compacted specimens at 140°F (60°C) different sets of specimens were tested at different temperatures as given in Table IX. Stabilities for the 2.0 in (51.0 mm) thick gyratory compacted specimens were corrected to the standard specimen thickness of 2.50 in (63.0 mm) using the stability correlation ratios presented in the standard test procedure.

Resilient Modulus

A diametral resilient modulus apparatus similar to that developed by Schmidt (42) was constructed for use in this study. A photograph of the apparatus is shown in Figure 11. This equipment consisted of an air actuated loading system capable of applying a 75.0 lb (3.4 x 10^2 N) pulsating load having a duration of 0.10 second every 3.0 seconds. A 300 lb (1.35 x 10^3 N) capacity load cell sensed the pulsated load applied across the diameter of a test specimen. The horizontal response (lateral deformation of the specimen) to the applied load was measured by a pair of linear variable transducers mounted in a yoke which was clamped to the specimen. A two-channel strip chart recorder monitored

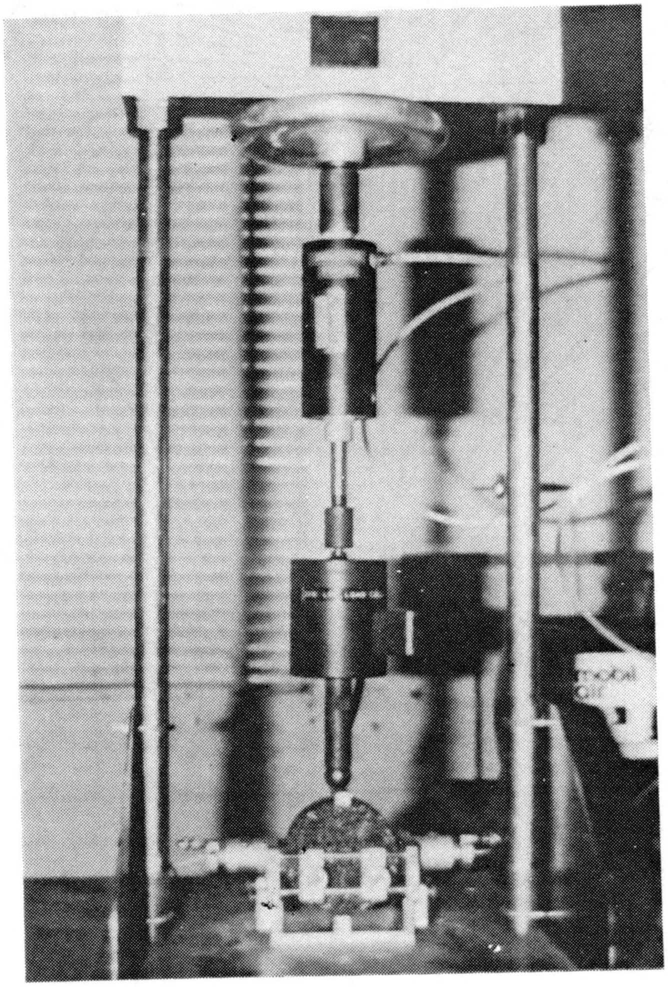


Figure 11. A Foamix Specimen During Resilient Modulus Testing

the output of the load cell and the amplified output of the deformation transducers. Details of this equipment and the calibration techniques used are presented in Appendix C.

Procedure. The yoke containing the diametrically opposed transducers was placed on the specimen holder with the transducers tips retracted. A 4.0 in (51.0 mm) diameter specimen was inserted through the yoke and centered on its side on the bottom loading strip. The rubber faced clamping screws on the yoke were then gently tightened to properly position the specimen. The top loading strip was centered 180^0 opposite the bottom loading strip and the yoke and holder assembly centered under the loading head. The loading shaft was then adjusted to make firm contact with the top loading strip. After separately adjusting each transducer until the tip contacted the specimen side and a reading of 1000 microinches was indicated by the recorder pen, the specimen was ready for testing.

The loading system of the apparatus was adjusted until the desired peak load of 75 lb (3.4×10^2 N) was indicated by the recorder. The specimen was then conditioned by 100 load applications (42) with the recorder set at a low chart speed (1.0 in per minute). Then the load and deformation was recorded at a higher chart speed (10.0 in per minute) during three or more loading cycles. The test was repeated after the specimen was rotated 90^0 in the yoke and holder assembly and the peak deformations compared. Agreement of the two deflection readings on the same specimen within 20 percent was desired.

The resilient modulus of a specimen was calculated from test results as follows:

$$M_R = \frac{P(\mu + .2734)}{t \cdot \Delta h}$$

where:

M_R = Resilient Modulus, psi.

P = Peak value of applied load, lb.

μ = Poisson's ratio (assumed value = 0.35).

t = Specimen thickness, in.

Δh = Average of lateral deformation peak values, in.

The resilient modulus was determined for compacted foamix specimens at temperatures of $30^{o}F$ ($-1^{o}C$), $78^{o}F$ ($25^{o}C$), and $105^{o}F$ ($40^{o}C$). It was considered that these temperatures represented a probable range experienced by asphalt base courses in Oklahoma.

Shear Strength

The shear strength of the respective foamed asphalt-fine aggregate mixtures was determined using an adaptation of the method and apparatus that has been standardized for field measurement of the shear strength of cohesive soils, i.e., the Field Vane Shear Test, ASTM D2573 (41). The use of such a test on asphalt cold mixes was initiated by Marias (43), who developed an apparatus that could be used in the field and laboratory to control the compaction of cutback-sand mixtures. Acott (8) also used the same apparatus and procedure to measure the shear strengths of foamed asphalt and sand base course mixtures. A vane shear device of similar function to the apparatus used by Marias and Acott was designed and then constructed by the OSU College of Engineering's Research and Development Laboratory. A photograph of this apparatus being used to test a compacted sample is shown in Figure 12. As shown, the

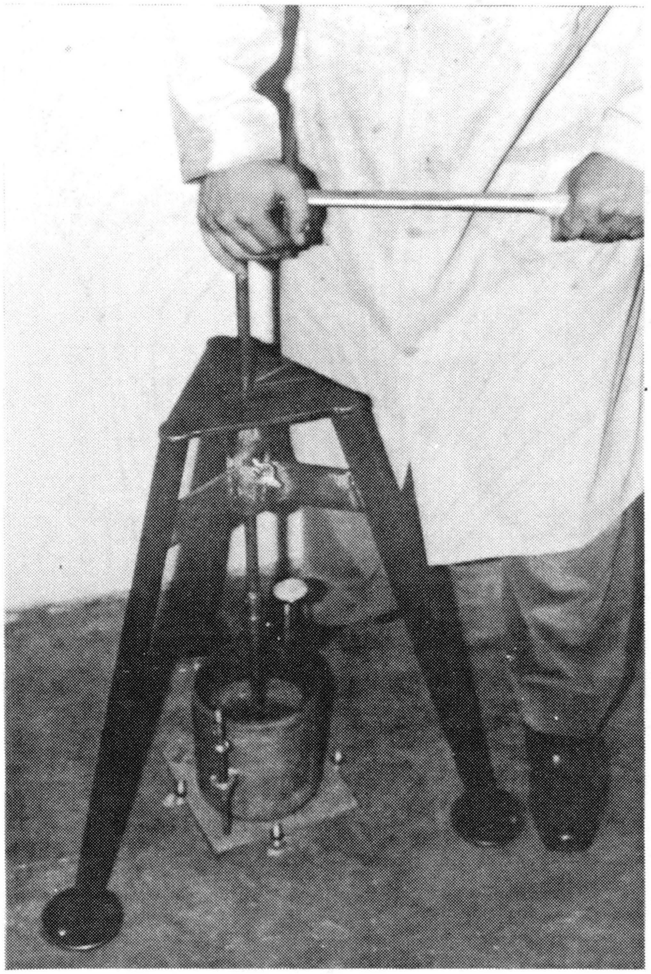


Figure 12. Vane Shear Testing Assembly
Used in Laboratory
Testing

device consisted of a tripod stand to hold the vane shaft perpendicular to the surface of the sample or material being tested; a four-bladed vane attached to the end of the shaft was driven into the sample; and a dial reading torque wrench used to apply and measure the torque required to shear the material being tested. A more detailed description of the apparatus and the dimensions of the cutter vanes are presented in Appendix D.

Procedure. Laboratory testing for shear stress of the compacted foamixes necessitated bolting the tripod stand to the concrete laboratory floor to maintain stability and alignment of the vane shaft. Bolt anchors were also positioned to hold the mold base plate centered beneath the stand. Following compaction and curing of a foamed asphalt mixture in the CBR mold (freshly molded specimens were dried while in mold at a temperature of 120^0F (49^0C) until they reached the desired curing condition), the specimen in the mold was positioned and the bolt anchors tightened. A size of vane cutter was selected for an estimated range of shear strength of the material, attached to the shaft and the shaft anvil placed on top of the shaft. The vane was driven into the compacted specimen until the top was slightly below the specimen surface. The torque wrench was attached to the vane shaft and a steady torque applied until failure occurred. The maximum torque reading on the dial was recorded. Procedural details and the necessary calculations for determining the shear stress are presented in Appendix D.

Although not used for field measurements in this study, it is anticipated that this apparatus could be used to control construction operations involving foamix base courses, as Marias did on cutback-sand mixtures. Proper correlation of field and laboratory results would have to be made before employing it in the field.

Moisture Exposure

The potential effects of moisture intrusion on the properties of molded foamix specimens were studied. Compacted specimens were long cured and then exposed to moisture following a procedure published by the Asphalt Institute for emulsion mixes (13). The equipment used consisted of a glass vacuum dessicator jar from which all dessicant had been removed and a vacuum system capable of evacuating the dessicator to 4.0 in (100.0 mm) Hg absolute pressure.

Although this soaking procedure was desirable from the standpoint of testing time, it was so severe on some foamix specimens made of aggregates (N-1) and (B-1), with less than 4.0 percent asphalt content, that the specimens disintegrated during moisture exposure. This drawback was overcome by wrapping the sides of all foamix specimens made from these aggregates with 2 in (51 mm) wide, 13 in (330 mm) long strips of polyethelene before they were exposed to moisture.

Procedure. The compacted specimens were weighed and placed in the vacuum dessicator. The specimens were then covered with 0.50 cm (13 mm) of water at ambient temperature, the dessicator lid was placed on the jar and connected to the vacuum system. The dessicator was evacuated to 4.0 in (100.0 mm) Hg absolute pressure for 60.0 ± 5.0 minutes before the vacuum was released and the specimens were then allowed to soak in water for another 60.0 ± 5.0 minutes. Soaked specimens were removed from the dessicator, surface dried and reweighed before subsequent testing for resilient modulus and Hveem resistance and stability.

The moisture change of the specimen was calculated as follows:

$$P = \frac{W_B - W_A}{W_B}$$

where:

P = Percent moisture increase.

W_B = Weight of specimen before soaking.

W_A = Weight of specimen after soaking.

CHAPTER V

RESULTS AND DISCUSSION

Foamix Preparation

The weight of asphalt cement that adhered to the mixer's blade and bowl during mixing and the dry aggregate batch weights required for preparing three gyratory-shear molded specimens were determined for each of the five aggregates, (N-1), (B-1), (S-1), (C-1) and (C-2). Table X gives the weight of asphalt cement lost during the preparation of foamed asphalt aggregate batches containing three levels or weights of asphalt cement. These levels of asphalt content were: 105, 150 and 195 grams and were obtained using settings of 3.1, 4.4 and 5.7 seconds on the dispenser's time controller. The data in Table X indicate that the greater the weight of foamed asphalt injected into a batch of fine aggregate, the larger the amount of asphalt cement adhering to the side of the bowl and blade. It is also evident from the data in Table X that the weight of asphalt cement adhering to the bowl and blade is not independent of the type of fine aggregate employed.

The weights of dry aggregate batches (for the five aggregates) required to prepare three [2.0 in (51.0 mm) height by 4.0 in. (102.0 mm) diameter] specimens are given in Table XI . If additional or control testing of the mixes is desired, an additional 100 grams of dry aggregates should be added to these weights. The data in Table XI suggests that the level of asphalt content is not significant in determining the

TABLE X

WEIGHT OF ASPHALT ADHERING TO MIXER BLADE AND BOWL

Delivered Foamed Asphalt Weight, Grams	Dispenser Time Controller Setting, Seconds	Weight of Asphalt Lost, in Grams, for Different Aggregate Types				
		(N-1)	(S-1)	(B-1)	(C-1)	(C-2)
105.0	3.1	21.10	20.5	19.8	16.10	14.3
150.0	4.4	26.7	27.0	24.1	20.8	18.1
195.0	5.7	34.2	33.8	32.4	27.6	25.2

TABLE XI

DRY AGGREGATE BATCH WEIGHTS FOR PREPARING THREE GYRATORY COMPACTED FOAMIX SPECIMENS

Aggregate Type	Level of Foamed Asphalt Content, % By Dry Weight of Aggregate				Mean Batch Weight ($\bar{x}$)	Standard Deviation (S)	Confidence Interval $\bar{x}$ ± 2S
	2.5 to 3.4	3.5 to 4.4	4.5 to 5.4	5.5 to 6.5			
	Batch Weights, Grams						
(N-1)	2160.0	2150.0	2160.0	2130.0	2150.0	14.0	2150.0 ± 28.0
(S-1)	2240.0	2250.0	2250.0	2270.0	2253.0	12.5	2253.0 ± 25.0
(B-1)	2340.0	2350.0	2375.0	2345.0	2352.0	16.0	2352.0 ± 32.0
(C-1)	2860.0	2850.0	2760.0	2740.0	2803.0	61.0	2803.0 ±122.0
(C-2)	2500.0	2520.0	2480.0	2510.0	2503.0	17.0	2503.0 ± 34.0

dry weight of the aggregate batch for a given aggregate type. Mean weight of batches ($\bar{x}$), the standard deviation (S) and the confidence interval for batch weights (independent of the asphalt content level) are given in the last 3 columns of Table XI . As expected, the mean batch weight ($\bar{x}$) is different from one aggregate to another with the greatest weight required for aggregate (C-1) and the lowest weight for aggregate (N-1). These differences are probably due to the general physical properties of the aggregates, i.e., specific gravity, gradation and particle index which is a measure of particle size, shape and surface texture.

Premolding Moisture Content

The optimum premolding moisture content is defined as the moisture content of the foamed asphalt-aggregate mixture at which the maximum density of the compacted mixture is obtained using the gyratory-shear method of compaction. The Statistical Analysis System (SAS) Computer program (44) was used to develop a multilinear regression model relating the premolding moisture content of the foamix and the influencing variables. It was thought that the variables that might affect the premolding moisture content were; the aggregate optimum moisture content, the percent material passing the No. 200 sieve, the percent asphalt cement content and the fluff point (see Chapter III). The following model form was used to develop a relationship between the foamix premolding moisture content and these variables:

$$MMC = \beta_0 + \beta_1(OMC) + \beta_2(PF) + \beta_3(FP) + \beta_4(AC)$$

where:

MMC = Foamix optimum premolding moisture content, percent by dry weight of aggregate;

OMC = Aggregate optimum moisture content, percent,

PF = Percent material passing sieve No. 200,

FP = Fluff point, percent,

AC = Asphalt content percent by dry weight of aggregates, and

β_0, β_1, β_2, β_3 and β_4 = the regression constants.

A stepwise regression technique provided by SAS was employed to develop the best representative model. The following model has a coefficient of determination (R^2) of 97.7 percent and observed significance level values less than 0.001 for all regression constants.

$$MMC = - 8.92 + 1.48 (OMC) + 0.40 (PF) - 0.39 (AC) \tag{5.1}$$

The relation given by Equation (5.1), suggests that, for a given aggregate used in a foamix, the higher the asphalt content the lower is the premolding moisture content. It should be noticed that the fluff point (FP) does not appear as an independent variable in the equation. When the fluff point was included as a variable, the effect of aggregate optimum moisture content was distorted, i.e., the observed significance level of the regression constant β_1 was higher than any customary significance level commonly used (0.05). When the fluff point (FP) was neglected, the stepwise regression analysis resulted in Equation (5.1). Similar results could be obtained when the optimum moisture content (OMC), rather than the fluff point (FP), was not included which suggested a direct association of the two variables (FP) and (OMC). It was decided to exclude the (FP) variable since the determination of the fluff point was not a standard test procedure.

Table XII presents the premolding moisture content (MMC) values determined experimentally together with the values computed by Equation (5.1). No apparent differences between the determined and the computed

TABLE XII

DETERMINED AND COMPUTED PREMOLDING MOISTURE CONTENTS FOR FOAMIXES

Aggregate*	Asphalt Content, %	MMC % Dry Aggregate Weight		Computed MMC As Percent of OMC
		Experimental	Computed	
(N - 1)	3.0	10.8	10.76	98
	3.75	10.5	10.47	95
	4.5	10.15	10.18	92.5
	5.25	9.85	9.85	89.5
(B - 1)	3.0	6.75	7.08	71.0
	4.0	6.25	6.70	67.0
	5.0	5.75	6.3	63.0
(S - 1)	3.0	6.75	6.85	61.0
	4.0	6.50	6.45	57.7
	5.0	6.15	6.07	53.7
(C - 1)	2.5	6.75	6.5	66.3
	4.0	6.35	5.97	60.4
	5.5	5.70	5.35	54.6
(C - 2)	3.0	6.8	6.83	67.8
	4.0	6.5	6.44	63.8
	5.0	6.2	6.06	60.0

*OMC and PF values for the five aggregates were presented in Table VII.

values are observed.

The optimum mixing moisture content, i.e., the moisture content of a fine aggregate required to achieve the best or most uniform distribution of foamed asphalt in the mix, was higher than the premolding moisture content of the mixture for all but one of the five aggregates evaluated in this study. The exception was the coarse (S-1) aggregate, for which good distribution of the foamed asphalt could not be obtained at either higher or lower moisture contents. This suggests that there is an interaction between the optimum mixing moisture content of an aggregate and the amount of passing No. 200 material contained in the aggregate. It also substantiates the selective ability of foamed asphalt for mixing with wet fines found by previous investigators (6, 10, 19).

Therefore, the optimum mixing moisture content can also be defined as the moisture content required to reach the best distribution of wet fines in an aggregate batch prior to mixing with foamed asphalt. The larger the percent fines (material passing No. 200 sieve) in the aggregate, the higher the potential for improved distribution of asphalt cement in the mix with increased moisture content. However, compacting a foa-mixture at its optimum mixing moisture content may result in lower density and strength values. While oven drying of the mixtures in the laboratory (aeration in the field) could be used to bring the mixes to their optimum premolding moisture content prior to compaction, this was considered excessively time consuming and not warranted for the aggregates being studied. Since the optimum mixing moisture contents of the fine aggregates in this study were only 10 to 20 percent higher than the premolding moisture contents, the optimum premolding moisture contents were employed for both mixing and molding.

Effect of Aggregate Particle Index and Percent Fines

Compacted specimens of the five foamixes containing four different asphalt contents (ranging from 2.5 to 5.5 percent by dry weight of aggregate) were tested for their Marshall stability values at temperatures of 78°F and 105°F (25°C and 40°C). Similarly, Marshall stability values were determined for test specimens of "hot mix" at these same test temperatures. The hot mix specimens were made from the same aggregates as used in the foamixes and contained 3, 4, and 5 percent asphalt. Four different factorial experiments were designed to statistically analyze the data and the variable relationships from these stability tests. Two of these factorial experiments (one for foamixes and the other for hot mixes) were used to study the effect of particle index on the strength of the asphalt treated aggregates. The other two factorial experiments studied the effects of percent fines (material passing the No. 200 sieve) on the Marshall stability of foamixes and hot mixes.

SAS computer program was used to conduct tests for evidence of real differences in the observed values. The results of these tests indicated the observed significance level and acceptance or rejection of the null hypothesis (no difference) based on a customary significance level of 0.05.

Particle Index

Marshall stabilities of molded specimens of foamixes made from aggregates (B-1), (C-2) and (C-1), with respective particle index values of 5.8, 12.9 and 15.5, and nearly the same percentage of fines (4.7 to 5.9 percent), were analyzed in a 3 x 4 x 2 completely randomized design factorial experiment. The first factor in the experiment consisted of the three levels of particle index. Asphalt contents of 2.5, 3.5, 4.5 and 5.5 percent by dry weight of aggregate comprised the second factor and testing temperature of 78 and 105°F (25 and 40°C) represented the third factor.

The analysis of variance of this experiment showed that the null hypothesis of no differences between mean stabilities was rejected. The values of the observed significance levels were less than 0.0001 for all main effects as well as for all interactions between factors. These significant interactions indicate that the factors under consideration, i.e., particle index, percent asphalt content and testing temperature, are not independent of each other and that conclusions related to differences in stabilities at different levels of particle index should be based on individual combinations of percent asphalt content and testing temperature. The Tukey's w procedure (45) was used for judging the significance of differences between paired stability means based on a significance level of 0.05.

Table XIII, presents the results of these pairwise comparisons between the three levels of particle index at different asphalt content-testing temperature combinations. The Marshall stabilities, of the foamixes, versus percent asphalt contents for the three levels of particle index at each of the two testing temperatures are illustrated in Figure 13. Foamixes with higher particle index values tend to produce higher Marshall stabilities, at the same percent asphalt content and testing temperature. The only case that deviated from this general trend was that at the combination of 5.5 percent asphalt content and 78°F (25°C) testing temperature for mixes (C-1) and (C-2). The Marshall stability of the foamix (C-2), with a particle index of 12.9, was

TABLE XIII

MULTIPLE COMPARISONS OF MEAN STABILITIES FOR PAIRED LEVELS OF PARTICLE INDEX (FOAMIXES)

Testing Temperature	Asphalt Content Percent	Sign of Paired Means Difference and the Results of Test of Significance of Real Difference		
		$[M_{C-1} - M_{C-2}]$*	$[M_{C-1} - M_{B-1}]$	$[M_{C-2} - M_{B-1}]$
78°F	2.5	(+) Significant**	(+) Significant	(+) Significant
	3.5	(+) Significant	(+) Significant	(+) Significant
	4.5	(+) Non-Significant**	(+) Significant	(+) Significant
	5.5	(-) Non-Significant	(+) Significant	(+) Significant
105°F	2.5	(+) Significant	(+) Significant	(+) Significant
	3.5	(+) Significant	(+) Significant	(+) Significant
	4.5	(+) Significant	(+) Significant	(+) Significant
	5.5	(+) Significant	(+) Significant	(+) Significant

*M_{C-1}, etc., is mean stability value for foamix (C - 1).

**Level of significance $\hat{\alpha}$ = .05

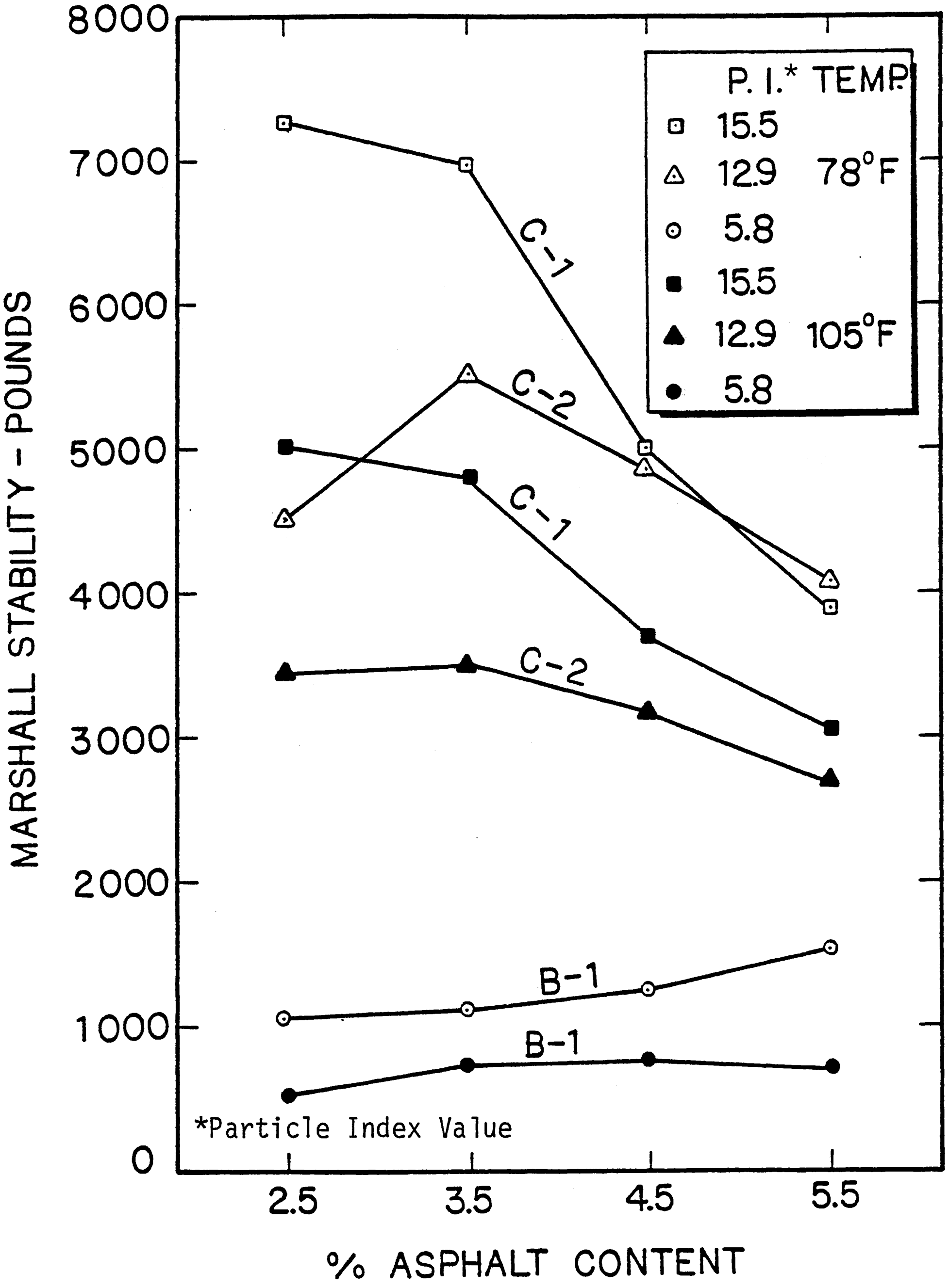


Figure 13. Effect of Particle Index on Marshall Stability of Foamixes

slightly higher than that of foamix (C-1), which had a particle index of 15.5.

This deviation is considered to be a minor one since both foamixes (C-1) and (C-2) have relatively high particle index values and, at the 78^{O}F (25^{O}C) test temperature, the difference between mean stabilities [MC-1 and MC-2] of the two mixes is non-significant for asphalt contents of 4.5 and 5.5 percent (see Table XIII). This implies that the stabilities of these two foamixes were statistically equal at these combinations of particle index, asphalt content and test temperature.

Also, it is widely accepted by other investigators (6, 10) that foamed asphalt has the ability to mix with and selectively coat the fines (material passing the No. 200 sieve) contained in an aggregate. The fine material functions as a filler for the asphalt in the mix and this filled asphalt forms a mortar between the larger aggregate particles. These mineral fines serve to drastically increase the viscosity of the asphalt binder and this provides increased strength to the compacted mixture. When the amount of asphalt is in excess of that needed to blend with the fines in a foamix, separate globules of more or less pure asphalt are present in the mixture. This unblended lower viscosity asphalt acts more as a lubricant than as a binding agent and will tend to reduce the strength or stability of the mix.

Since the asphalt content, at which the highest stability of foamix (C-1) is achieved, is one percent less than for foamix (C-2) at a test temperature of 78^{O}F (25^{O}C), it is likely that, at 5.5 percent asphalt content, the excess unblended asphalt in foamix (C-1) was enough to reduce the Marshall stability below that of foamix (C-2), overcoming the relative effect of a higher particle index.

A second factorial experiment was designed to study the effect of particle index on the Marshall stability of compacted hot mixes made with the same aggregates used in the foamixes, i.e., aggregates (B-1), (C-2) and (C-1). This experiment was similar to that for the foamixes with the exception that the levels of asphalt content were 3.0, 4.0 and 5.0 percent. F-test from the analysis of variance showed the significance of all treatments main effects as well as interactions between temperature and asphalt content and between temperature and particle index.

Figure 14 illustrates the relationship between Marshall stability and the percent asphalt content at different levels of particle index and testing temperature for the hot mixes. Generally, the Marshall stabilities for the respective combinations are considerably lower for the hot mixes than for the foamixes within a similar range of asphalt contents (compare Figure 14 and Figure 13). It is also evident from Figure 14 that stabilities of the hot mixes, at all levels of particle index are higher when measured at 78^{o}F (25^{o}C). This, no doubt, is a reflection of decreased viscosity in the asphalt binder at the higher test temperature. The results in Table XIV show that the differences between mean stabilities at paired levels of particle index are only significant in 4 out of 18 combinations of particle index, asphalt content and testing temperature. This suggests that, within the limits of this study, the stabilities of compacted hot mixed aggregates are more dependent on testing temperature than on the level of the aggregate's particle index. This propensity is different from that of the foamixes where the aggregate's particle index is the predominant factor influencing stability.

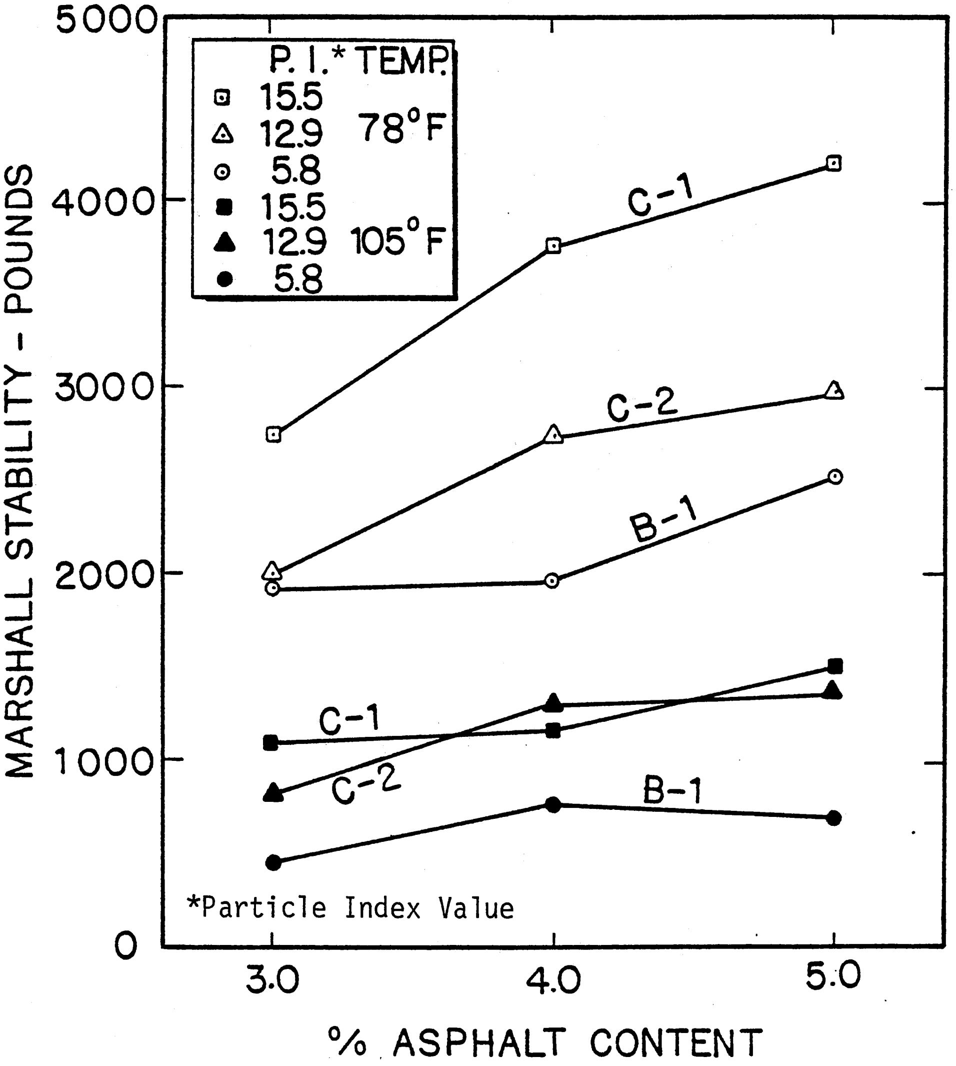


Figure 14. Effect of Particle Index on Marshall Stability for Hot Mixes

TABLE XIV

MULTIPLE COMPARISONS OF MEAN STABILITIES FOR PAIRED LEVELS OF PARTICLE INDEX (HOT MIXES)

Testing Temperature	Asphalt Content Percent	Sign of Paired Means Difference and the Result of Test of Significance of Real Difference		
		$[M_{C-1} - M_{C-2}]$*	$[M_{C-1} - M_{B-1}]$	$[M_{C-2} - M_{B-1}]$
78^{o}F	3.0	(+) Non-Significant**	(+) Non-Significant	(+) Non-Significant
	4.0	(+) Non-Significant	(+) Non-Significant	(+) Non-Significant
	5.0	(+) Significant**	(+) Significant	(+) Significant
105^{o}F	3.0	(+) Non-Significant	(+) Significant	(+) Significant
	4.0	(+) Non-Significant	(+) Non-Significant	(+) Non-Significant
	5.0	(+) Non-Significant	(+) Non-Significant	(+) Non-Significant

*M_{C-1}, etc., is mean stability value for foamix (C - 1).

**Level of significance $\hat{\alpha}$ = .05.

Percent Fines

To determine the effect of percent fines (material passing the No. 200 sieve) on the Marshall stability of compacted foamixes and hot mixes, aggregates (S-1), (B-1) and (N-1) were used. These three aggregates had roughly the same particle index values, ranging from 5.3 to 6.3, and respectively contained 0.50, 6.0 and 11.4 percent (by dry weight of aggregate) material passing the No. 200 sieve.

The relationship between Marshall stability and percent asphalt content at different levels of percent fines and testing temperature for the foamixes and hot mixes are shown respectively in Figures 15 and 16. At the same test temperature, Figure 15 suggests that the higher the percent fines (material passing No. 200 sieve) in a foamix the higher the Marshall stability value at any level of asphalt content. For example, at a testing temperature of $78^{o}F$ ($25^{o}C$), the Marshall stability values of specimens made from foamix (N-1) [with 11.4 percent fines] is higher than those made from foamix (B-1) [with 6.0 percent fines]. Also, the Marshall stability values of specimens made from foamix (S-1) [with 0.5 percent fines] are lower than those of (B-1) and (N-1). This trend is also applicable at a testing temperature of $105^{o}F$. Figure 16 (hot mixes) indicates the dominating influence of the testing temperature in affecting the Marshall stability values of hot mixes. The Marshall stabilities of specimens made from hot mixes (N-1), (B-1) and (S-1) determined at the low testing temperature of $78^{o}F$ ($25^{o}C$) are higher than those determined at the high testing temperature of $105^{o}F$ ($40^{o}C$). Furthermore, at a testing temperature of $78^{o}F$ ($25^{o}C$), the Marshall stability values of specimens made from hot mix (S-1) [0.5 percent fines] are higher than those from hot mixes (N-1) and (B-1). This is most likely due to the better coating of

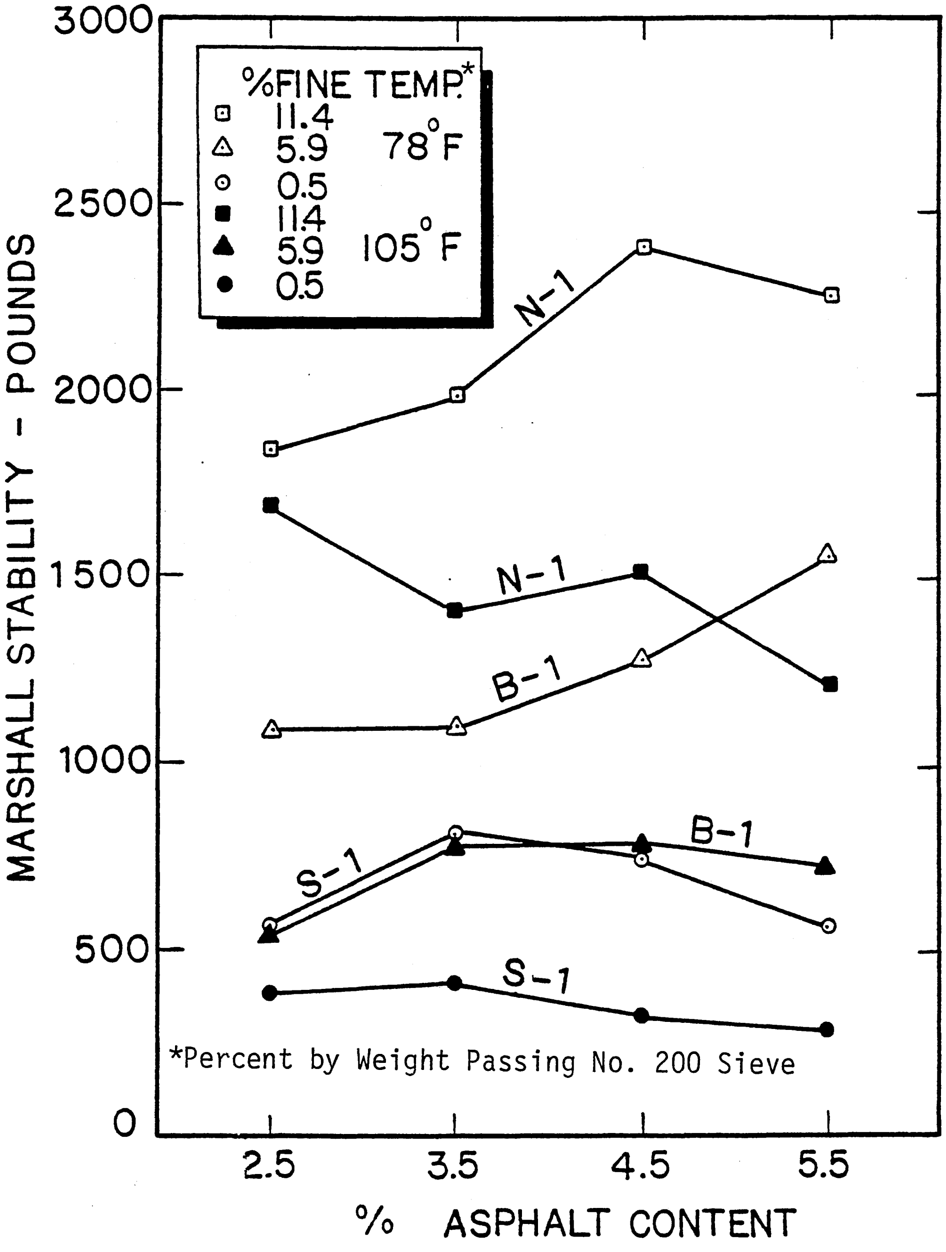


Figure 15. Effect of Percent Fines (Passing No. 200 Sieve) on Marshall Stability for Foamixes

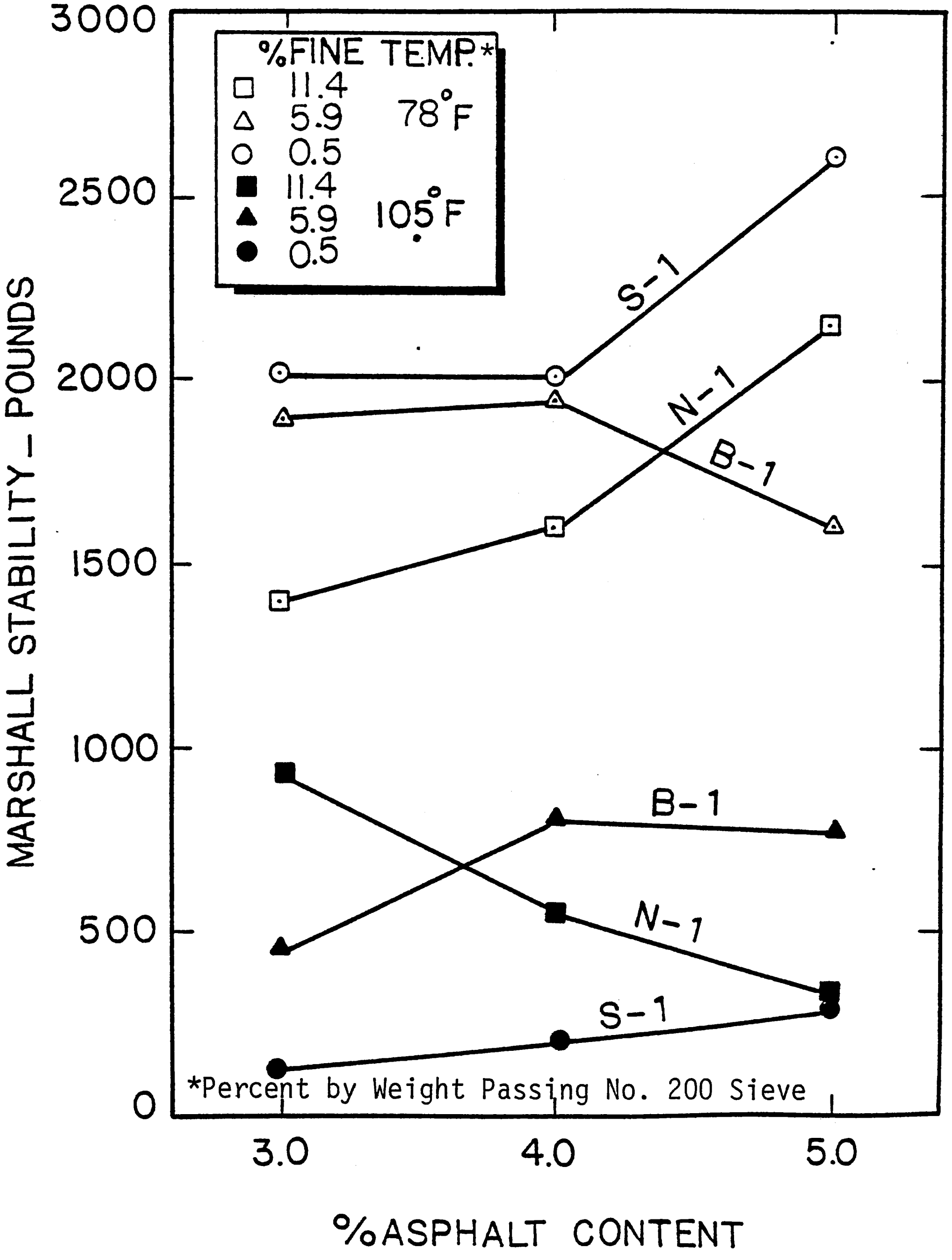


Figure 16. Effect of Percent Fines (Passing No. 200 Sieve) on Marshall Stability for Hot Mixes

hot asphalt cement for aggregate (S-1) which had the least surface area as compared to the other two aggregates.

The analysis of variance of the effect of percent fines (material passing No. 200 sieve) on the stability values of compacted specimens from foamixes and hot mixes implied conclusions similar to that obtained for the particle index analysis discussed in previous section. All main effects and interactions between treatments levels for the foamixes were significant at a significance level of 0.05. For the hot mixes experiment, only the three-factor interaction was not significant. Therefore, multiple comparisons between mean stabilities of paired levels of percent fines at all combinations of asphalt content and testing temperature were computed using the Tuckey's w Procedure. Tables XV and XVI presents the results of the multiple comparisons for foamixes and hot mixes, respectively.

The results of multiple comparisons for the foamixes, Table XV , indicate that the differences between mean stability values of every paired level of percent fines are significant at all combinations of asphalt content and test temperature. Furthermore, all differences have positive signs, i.e., the mean stabilities for specimens made from foamix (N-1) [11.4 percent fines] are significantly higher than those for specimens made from foamixes (B-1) and (S-1) [6.0 and 0.5 percent fines, respectively]. Also, the mean stabilities of specimens made from foamix (B-1) are significantly higher than those from foamix (S-1). Therefore, a larger percentage of materials passing sieve No. 200 in a foamix would result in higher Marshall stability values.

For hot mixes, the differences between mean stabilities of paired levels of the percent fines factor, were non-significant for all levels

TABLE XV

MULTIPLE COMPARISONS OF MEAN STABILITIES FOR PAIRED LEVELS OF PERCENT FINES (FOAMIXES)

Testing Temperature	Asphalt Content Percent	Sign of Paired Means Difference and the Results of Test of Significance of Real Difference		
		$[M_{N-1} - M_{B-1}]$	$[M_{N-1} - M_{S-1}]$	$[M_{B-1} - M_{S-1}]$
78°F	2.5	(+) Significant	(+) Significant	(+) Significant
	3.5	(+) Significant**	(+) Significant	(+) Significant
	4.5	(+) Significant	(+) Significant	(+) Significant
	5.5	(+) Significant	(+) Significant	(+) Significant
105°F	2.5	(+) Significant	(+) Significant	(+) Non-Significant
	3.5	(+) Significant	(+) Significant	(+) Significant
	4.5	(+) Significant	(+) Significant	(+) Significant
	5.5	(+) Significant	(+) Significant	(+) Significant

*M_{C-1}, etc., is mean stability value for foamix (C - 1).

**Level of significance $\hat{\alpha}$ = .05.

of asphalt content (3.0, 4.0 and 5.0 percent) at a testing temperature of 78^{0}F (25^{0}C). The prevailing negative signs in the upper half of Table XVI [78^{0}F (25^{0}C)] support the data illustrated in Figure 16 which was discussed previously; however, these differences are not statistically significant. At a testing temperature of 105^{0}F (40^{0}C) [lower half of Table XVI] the positive signs of differences between mean stability values are dominating, i.e., the Marshall stabilities of compacted specimens from hot mixes made from aggregates with higher percentages of fines are greater than those made from aggregates with lower percentages fines.

In general, it is evident that the stability values of specimens made from hot mixes tend to be more dependent of the testing temperature than it is on the constituting aggregate properties. In preparing a hot mix, the asphalt cement was forced to coat most of the aggregate particles. This kind of mix structure forms a surface failure which is sensitive to temperature changes. The mix structure of a foamix is different from that of a hot mix. It is somewhat similar to that of a Portland cement concrete. The asphalt cement, while in the foam condition, is blended with fine materials forming a black strong mortar which fills the voids between larger aggregate particles. This mortar has proved to be less susceptible to temperature change and stronger than plain asphalt cement. This kind of mix structure (of a foamix) provides a greater mechanical interlock of the larger particles. Therefore, the angularity and surface texture of the aggregate play a more dominant role in predicting the strength of a foamix than a hot mix.

Effect of Curing

Compacted specimens of the respective foamixes were subjected to

TABLE XVI

MULTIPLE COMPARISONS OF MEAN STABILITIES FOR PAIRED LEVELS OF PERCENT FINES (HOT MIXES)

Testing Temperature	Asphalt Content Percent	Sign of Paired Means Difference and the Results of Test of Significance of Real Difference		
		$[M_{N-1} - M_{B-1}]$	$[M_{N-1} - M_{S-1}]$	$[M_{B-1} - M_{S-1}]$
78°F	3.0	(-) Non-Significant**	(-) Non-Significant	(-) Non-Significant
	4.0	(-) Non-Significant	(-) Non-Significant	(-) Non-Significant
	5.0	(+) Non-Significant	(-) Non-Significant	(-) Non-Significant
105°F	3.0	(+) Significant**	(+) Significant	(+) Significant
	4.0	(-) Non-Significant	(+) Non-Significant	(+) Significant
	5.0	(-) Non-Significant	(+) Non-Significant	(+) Non-Significant

*M_{C-1}, etc., is mean stability value for foamix (C - 1).

**Level of significance $\hat{\alpha}$ = .05.

three curing procedures or periods of drying to reduce the moisture in these specimens to levels below the molding moisture contents. Following the short, intermediate, and long curing processes (described in Chapter IV), these compacted specimens were tested to 1) verify that a loss in molding moisture content was accompanied by a gain in strength; 2) study some of the factors that affect the rate of curing; and 3) provide experience and information that might be used for field control and correlation on actual foamix construction projects. The effects of curing on the percent moisture lost, Hveem resistance and stability, and resilient modulus of the foamix specimens are shown in Figures 34 through 47 of Appendix E.

All foamix specimens lost approximately 50 percent of their molding moisture content during the 24 hour in-mold curing period at laboratory temperature. The moisture contents after long curing ranged from about 85 percent for foamix (C-2) to about 95 percent for foamix (N-1). However, the initial rate of curing, i.e., the loss of moisture content with time during in-mold exposure at lab temperature, was higher for all foamix specimens, with a lower rate of moisture loss evidenced during intermediate curing and a still lower rate during long curing.

The percent of curing of the specimens was affected by the asphalt content and the air voids contained in the compacted foamix. Foamix specimens with low asphalt contents lost higher percentages of their initial moisture than those with higher asphalt contents, although the rates of loss experienced during the respective curing periods and conditions remained nearly constant for a given type of foamix. Total evaporative moisture losses and rates of loss were generally lower for the compacted foamix specimens with lower air void contents [Foamixes (C-1) and (C-2)] than those having a higher percent of air voids [Foamixes

(N-1) and (B-1)] (see Figures 34 to 37, Appendix E and Table XVII).

The Hveem stability and resistance values of all the compacted foamixes increased with increased curing, although at considerably different rates between the respective levels of curing (see Figures 38 to 45, Appendix E). The greatest increase in strength values (above the values obtained with freshly molded specimens) occurred after short term curing. Only slight increases in stability and resistance were observed for oven curing beyond the first 24 hours (intermediate curing) in foamixes (N-1) and (B-1).

It should also be noted that specimens of these two foamixes, i.e., (N-1) and (B-1), had drastically lower stabilities after all levels of curing than did the other two foamixes. Apparently, the moderate strength gained after short and intermediate curing was not enough to sustain even the low dynamic type loads applied during the resilient modulus test and these specimens at all asphalt contents failed before values could be determined. However, increased resilient modulus values with increased level of curing were found for foamix specimens (C-1) and (C-2) (see Figures 46 and 47, Appendix E). The only consistent relation between asphalt content and resilient modulus over the curing range was exhibited by the (C-1) foamix specimens.

Curing also greatly improved the resistance of the foamixes to applied shear stress. The effect of the extent of curing on the vane shear stress measured on compacted specimens of the (N-1) and (B-1) foamixes at their respective design asphalt contents of 5.5 and 4.5 percent is shown in Figure 17. The shear resistance of these specimens after short curing is considered very low relative to that after intermediate curing. Recalling that the gyratory-shear size specimens of these foamixes exhibited

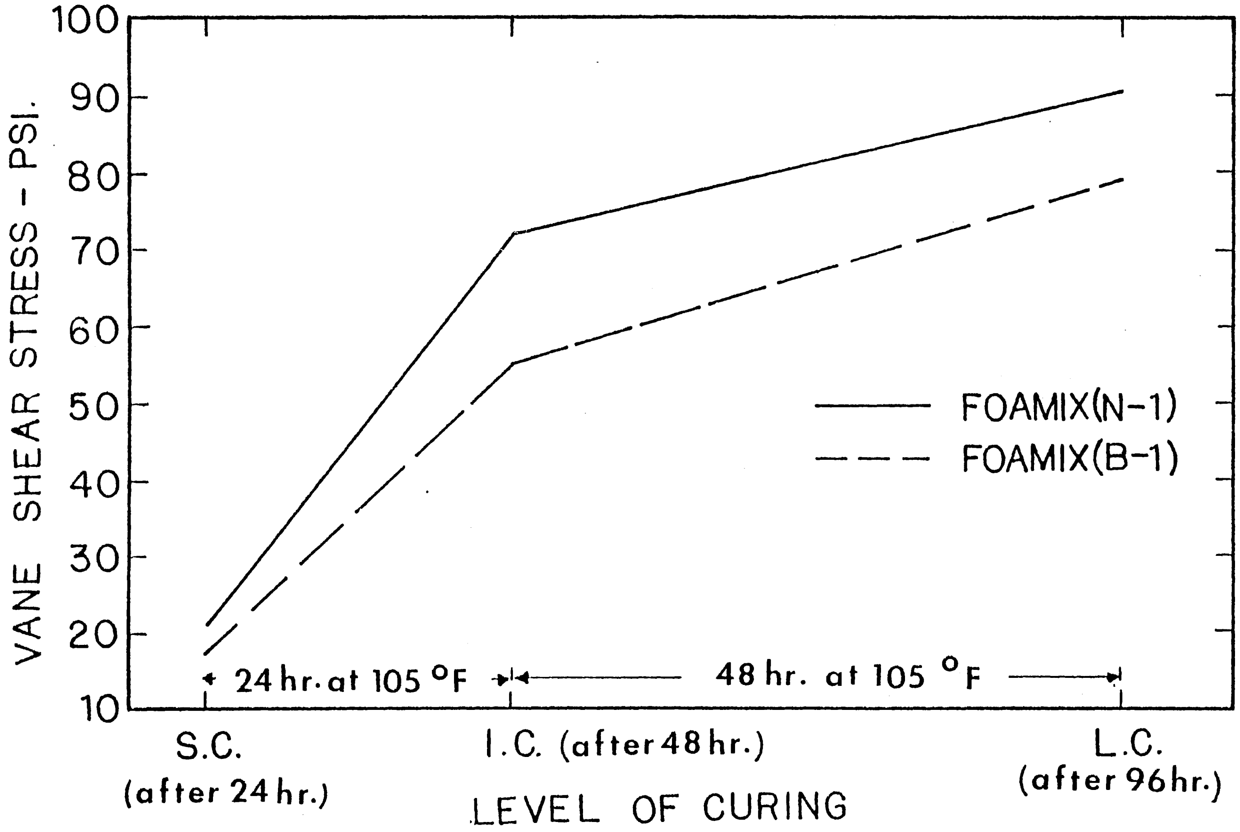


Figure 17. Effect of Curing Level on Vane Shear Stress

TABLE XVII

GENERAL PROPERTIES OF FOAMIXES

Foamix Type	Percent Asphalt Content	Dry Density, pcf	Percent Air Voids	Percent Moisture Absorbed	Percent Hveem Stability Loss*	Percent Hveem Resistance Loss*
(C - 1)	2.5	146.5	6.2	2.4	22.0	4.0
	3.5	146.8	4.6	1.75	15.0	3.0
	4.5	143.0	5.2	1.4	3.0	3.0
	5.5	142.5	4.7	1.3	1.0	3.0
(C - 2)	2.5	127.7	12.2	4.5	41.0	5.0
	3.5	129.5	9.8	3.4	30.0	3.0
	4.5	128.0	8.6	2.5	24.0	1.0
	5.5	130.5	6.7	2.2	23.0	1.0
(N - 1)	2.5	110.0	25.0	25.0	70.0	32.0
	3.5	110.2	23.5	16.2	64.0	29.0
	4.5	111.4	22.5	10.0	48.0	20.0
	5.5	110.0	22.0	5.7	27.0	10.0
(B - 1)	2.5	119.5	18.0	16.0	42.0	22.0
	3.5	120.5	16.2	12.8	40.0	20.0
	4.5	122.5	14.0	9.6	23.0	12.0
	5.5	121.8	13.7	2.8	20.0	3.0

$$\text{*Percent loss} = \frac{(\text{Dry Value} - \text{Wet Value}) \times 100}{\text{Dry Value}}$$

a marked increase in stability after short term curing, it was reasonable to expect a similar increase in resistance to applied shear in the 6.0 in diameter by 5.0 height specimens compacted and short cured for this test. The vane shear stress for foamixes (C-1) and (C-2) exceeded the ranges of the three available vane cutters. These materials had shear resistance greater than 150 psi (the upper range of shear that could be measured with the available vane cutters--see Figure 33, Appendix E). No subsequent vane shear testing on these foamixes was performed to avoid the probability of overstressing the vane cutters.

The larger vane shear test specimens were oven dried at 120^{0}F (49^{0}C) until they lost the same percentage of their molding moisture content as had the smaller gyratory-shear specimens following the short cure period. Apparently, there was a size or thickness effect such that, while the percentage of moisture loss was the same, the material near the surface of the exposed ends of the specimen was dried to a greater extent than the material a few inches below the surface. Thus, the degree of curing in the specimen was not uniform throughout the specimen and the shear strength measured at depth below the surface was reduced by the semi-cured foamix.

This phenomenon suggests an interaction between curing and the thickness of a foamix layer. In an actual paving project the upper or exposed side of a thick compacted foamix layer will gain strength (shear resistance) faster than the interior and/or bottom of the layer. Amount of curing and the corresponding rate of strength gain of foamixes in field applications are dependent on the mixture variables, as previously discussed, and those variables associated with the construction and environmental conditions.

Determination of Design Asphalt Content

The determination of the design asphalt contents of the four foamixes (N-1), (B-1), (C-1) and (C-2) was based on their values of Hveem stability, Hveem resistance, and in some cases, vane shear stress. Since the improvement in the characteristics of compacted foamixes after short and intermediate curing levels was considered to represent the pavement condition during a relatively short period of its life (two weeks), the design evaluation of the foamixes was based on their properties after long term curing.

The effect of 2.5 to 5.5 percent foamed asphalt on Hveem resistance, Hveem stability and resilient modulus for the four foamixes is illustrated in Figures 48 through 59 in Appendix E. The effect of temperature on the laboratory measured vane shear stresses of foamixes (N-1) and (B-1) at their design asphalt content is shown in Figure 60 (Appendix E). Table XVII lists the general physical properties of foamixes, i.e., percent air voids, percent moisture absorption and percent stability and resistance loss after moisture exposure.

Foamix (C-1) was superior to those of the other three foamixes, (C-2), (N-1) and (B-1). At low asphalt contents, foamixes (N-1) and (B-1) had a higher susceptibility to moisture intrusion than foamixes (C-1) and (C-2). Foamix (B-1) had greater density, lower percent air voids and lower percent moisture absorption than foamix (N-1). However, foamix (N-1) had better strength properties and less temperature susceptibility than foamix (B-1). Specimens made from foamix (B-1), at 5.5 percent asphalt content, showed a very high susceptibility to temperature. These specimens, at this asphalt content, failed during resilient modulus testing at 105°F (40°C). It is thought that failure occurred due to the presence of free asphalt cement not blended with fines in these specimens. Except for the above mentioned case all foamixes were not significantly affected by testing temperature. Values of Hveem stabilities, Hveem resistance and resilient modulus measured at 105°F (40°C) were not less than 70 percent of those determined at 78°F (25°C).

The design asphalt contents for the foamixes were based on the design criteria given in Table XVIII. These criteria are predicted on the experience gained in this study, the emulsified asphalt-aggregate design criteria used by several agencies nationwide (13) and a foamix criteria for low traffic volume roads recommended by Acott (46). Three other factors were considered in evolving this design criteria. 1) It is not practical to use hot mix design criteria and test procedures for foamixes without some modification, e.g., reducing the temperature for stability testing to 105°F (25°C). 2) Design requirements for pavements that carry light traffic should be lower than those for heavy traffic volumes. 3) Foamixes gain strength significantly with time or aging (6, 16).

The design foamed asphalt contents for foamixes (N-1), (B-1), (C-1) and (C-2) were 5.5, 4.5, 4.0 and 4.5 percent, respectively. Although foamix (C-1) could have been designed for a lower asphalt content (see Table XVII and Figures 50, 54, and 58 in Appendix E), a higher asphalt content was selected to improve its fatigue properties. A summary of the test results for the four foamixes at their design asphalt content is given in Table XIX.

The information in Tables XVIII and XIX suggests that foamixes (C-1) and (C-2) can be used as base course materials for heavy traffic

TABLE XVIII

DESIGN CRITERIA USED FOR EVALUATION OF FOAMIXES[e]

Test Method	Light[a] Traffic	Heavy[b] Traffic
Hveem Stability, 105°F (40°C)	N.A.	30.0 min.
Percent Stability[c] Loss 78°F (25°C)	N.A.	30.0 max.
Hveem Resistance, 105°F (40°C)	75.0 min.	90.0 min.
Percent Resistance[d] Loss, 78°F (25°C)	20.0 max.	10.0 max.
Vane Shear Strength, psi	30.0 min.	N.A.
Percent Moisture Absorbtion	10 max.	5.0 max.

[a] EAL $\leq$ 360,000

[b] EAL $\geq$ 360,000

[c] $$\frac{(\text{Dry Stability} - \text{Wet Stability})}{(\text{Dry Stability})} \times 100$$

[d] $$\frac{(\text{Dry Resistance} - \text{Wet Resistance})}{(\text{Dry Resistance})} \times 100$$

[e] All Values Determined After Long Term Curing

TABLE XIX

GENERAL PROPERTIES OF FOAMIXES AT THEIR DESIGN ASPHALT CONTENT

Test Method	Foamix (N - 1)	Foamix (B - 1)	Foamix (C - 1)	Foamix (C - 2)
Hveem Stability, 105°F (40°C)	—	—	63.0	64.0
Percent Stability Loss, 78°F (25°C)	—	—	8.0	25.0
Hveem Resistance, 105°F (40°C)	86.0	77.0	95.2	95.0
Percent Resistance Loss, 105°F (25°C)	7.0	15.0	3.0	1.0
Vane Shear Strength, psi	48.5	37.0	—	—
Percent Moisture Absorbtion	5.8	8.0	1.5	2.5

highways where the EAL is greater than 360,000 repetitions. Foamixes (N-1) and (B-1) are suitable for use as base course materials for light traffic volume roads. In this study a light traffic highway is defined as one that carries EAL less than 360,000. The EAL represents the total number of equivalent 18-kip single-axle loads estimated for the design lane during the design period.

CHAPTER VI

STRUCTURAL EVALUATION OF FOAMIXES

Design Principles

A mechanistic design procedure for a flexible pavement includes three major elements.

1. A method for the characterization of material properties.
2. A method of structural analysis for multilayered systems.
3. A method of relating mechanistic responses to distress.

Material Characterization

Three different concepts have been proposed to characterize paving materials; linear elastic, non-linear elastic and linear viscoelastic (46). For the purposes of this study the linear elasticity concept was used as the basic constitutive relationship for the evaluation of paving materials. These pavement materials, the ASSHTO Road Test surface course and four different foamixes, were characterized by a Poisson's ratio (μ) [assumed to be 0.35 for all asphalt treated materials] and a modulus of elasticity [also called resilient modulus, M_R] versus temperature curve. The subgrade materials were also characterized by a resilient modulus value and a Poisson's ratio of 0.45. A number of investigations (47, 48, 49) have shown that the subgrade resilient modulus changes, throughout the year, in the manner qualitatively illustrated in Figure 18. In this figure:

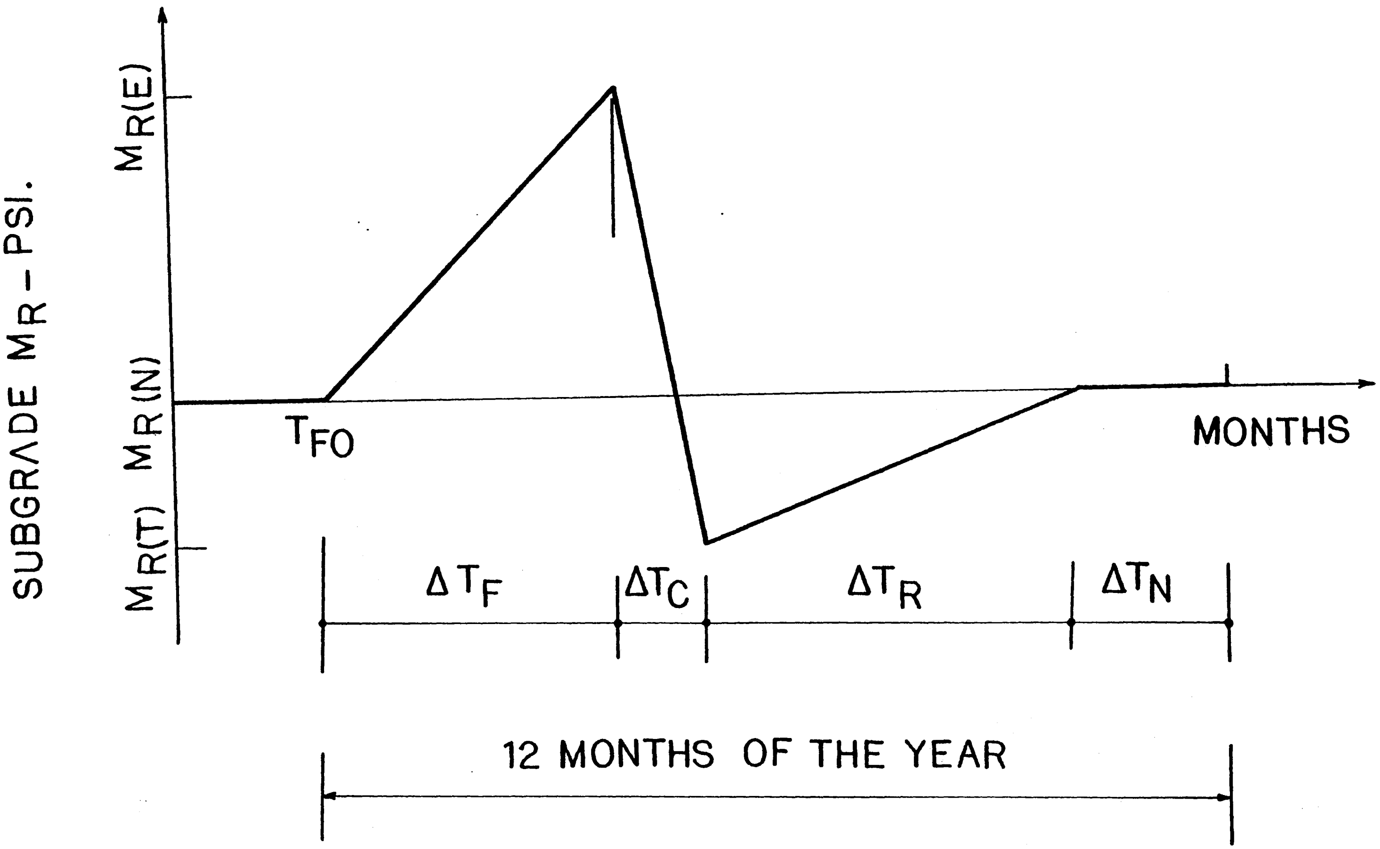


Figure 18. Qualitative Environmental Change of Subgrade Resilient Modulus Throughout the Year (After the Asphalt Institute, 49)

$M_R(F)$ = Frozen subgrade modulus

$M_R(N)$ = Normal subgrade modulus

$M_R(T)$ = Thaw (reduced) subgrade modulus

and,

T_{FO} = Month freeze starts

ΔT_F = Time of freeze in months

ΔT_C = Time of critical thaw in months

ΔT_R = Time of thaw recovery

ΔT_N = Time of normal subgrade condition

The actual magnitude of the changes and their durations are variable depending primarily on the environment, the groundwater conditions and the subgrade soil.

The characterization of the properties of all materials used in this study was based on the Oklahoma environmental conditions. Quantitative values of material characteristics, relative to Oklahoma weather conditions, are given in the following section.

Structural Analysis

A variety of computer programs are currently available for the solution of a boundary value problem for a multilayered pavement system. In general, these layered system computer programs use a matrix formulation to simplify the problem of determining the distribution of strains in a stratified semi-infinite elastic medium under the compressive action of a rigid body (50).

The computer program DAMA (49) was used for the multilayered elastic analysis of this study. The DAMA program is designed to obtain the required elastic solutions within a pavement that comprises a maximum of

5 layers (including a subgrade layer) under single or dual load conditions. Three transverse computational points are automatically specified within the computer program to fix the response locations. These computational points are at the center of one tire, at the edge of one tire, and at the midpoint of the dual system. Environmental effects are characterized by mean monthly air temperature and variable monthly material moduli. Subgrade resilient modulus values are input to the program on a monthly basis. This permits the assessment of the effects of subgrade freeze and thaw conditions throughout the year.

Design Criteria

The next design element is to compare the predicted strains in an assumed pavement structure with maximum allowable or limiting values. Two limiting design conditions were considered in this study. They are, the vertical compressive strain (ε_c) at the subgrade surface and the horizontal tensile strain (ε_t) on the underside of the asphalt treated layer. Limiting vertical subgrade strains were selected to minimize surface rutting caused by overstressing the subgrade. Maximum allowable load applications for a given horizontal tensile strain were used to minimize pavement cracking.

Subgrade Strain Criteria

The limiting vertical subgrade strain related to load repetitions is shown in Figure 24. The criteria are based on the following equation:

$$N = 1.365 \times 10^{-9} \times (\varepsilon_c)^{-4.477} \qquad (6.1)$$

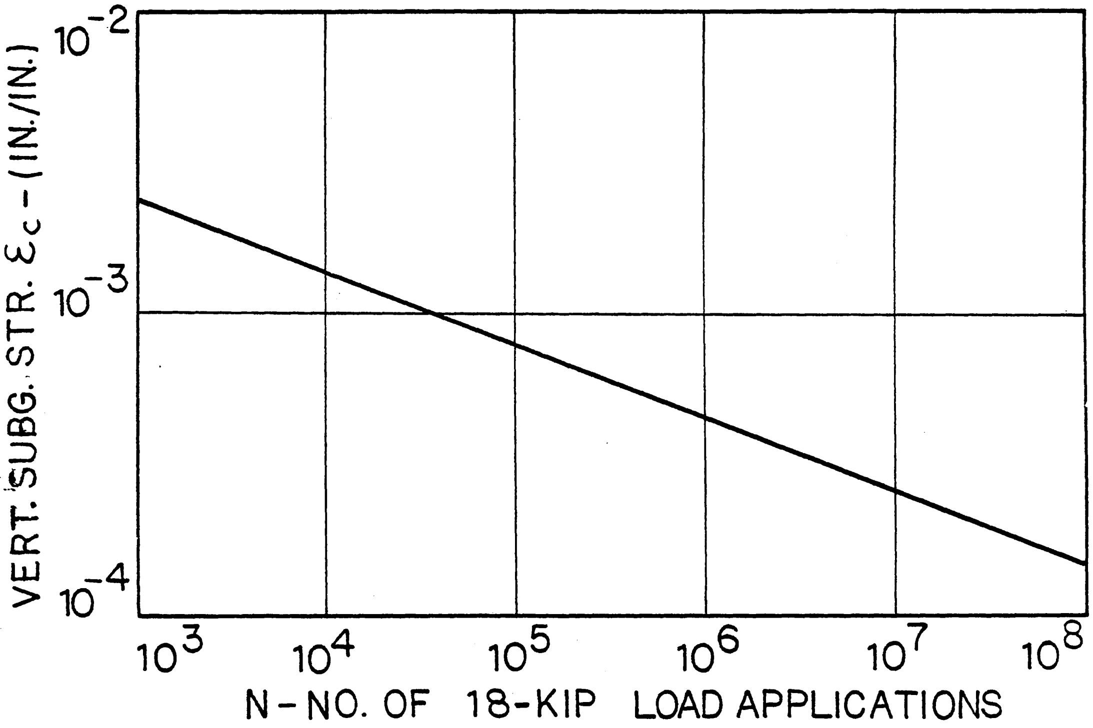


Figure 19. Subgrade Strain Criterion (After Santucci, 52)

where:

N = Number of 18,000 lb equivalent single axle load repetitions; and,

ε_c = Vertical compressive strain at subgrade surface (in./in.).

These limiting criteria were developed by Monismith (51) and were used by Santucci in the development of the Chevron thickness design procedure (52). The same criteria were also used by the Asphalt Institute (49) in developing the Ninth Edition of MS-1 (Thickness Design-Asphalt Pavements for Highways and Streets). Using these criteria the rut depth should not exceed approximately 0.5 in (13.0 mm) for the design traffic.

<u>Tensile Strain (Fatigue) Criteria</u>

Extensive research has indicated that fatigue damage for asphalt mixes can be described by the following relationship:

$$N = K(1/\varepsilon_t)^n \tag{6.2}$$

where:

N = Number of load repetitions to failure;

ε_t = Initial tensile strain (in./in.) determined for treated materials in constant stress type loading; and

K,n = Experimentally determined constants.

Fatigue criteria developed by Finn et al. (53) for NCHRP project 1-10B in which a field shift factor was developed to account for the improved fatigue properties of asphalt mixes is given in Equation (6.3).

$$N_f = 18.4\ (4.325 \times 10^{-3} \times (\varepsilon_t)^{-3.29} \times (M_R)^{-.854}) \tag{6.3}$$

where:

N_f = Number of 18,000 lb equivalent single axle load repetitions to failure;

ε_t = Tensile strain repeatedly applied;

M_R = Elastic or resilient modulus, psi.

The constant value 18.4 is the field to laboratory shift factor. This factor is related to the AASHTO Road Test Loop 4 and Loop 6 (9) where fatigue cracking of more than 45 percent of the wheel path area occurred. The parenthetically separated portion of the equation represents the fatigue curves, as found in the laboratory, for mixes made of the same materials and asphalt contents used in the AASHTO Road Test asphalt concrete. Equation (6.3) was modified to reflect the effect of both the percent by volume of air voids (V_v) and the percent by volume of asphalt cement (V_b) using a correction factor C defined as:

$$C = 10^M \tag{6.4}$$

where

$$M = 4.84 \left(\frac{V_b}{V_v + V_b} - 0.69\right)$$

The correction factor (C) was evolved by Santucci and used in the development of the Chevron design procedure (52). The Asphalt Institute, also, adopted the same model, Equation (6.3) with the correction factor (C), as the limiting fatigue criteria for both asphalt concrete and emulsified asphalt mixes (49).

Since laboratory fatigue testing of the foamixes was not performed in this study, the following fatigue model, the product of multiplying

Equation (6.4) by Equation (6.3),was adopted as an approximation representing the fatigue potential of the foamixes studied:

$$N_f = 0.3852 \left(\frac{V_b}{V_v + V_b} - 0.69\right)((\varepsilon_t)^{-3.291} \times (M_R)^{-.854}) \qquad (6.5)$$

The fatigue cracking models for the AASHTO asphalt concrete and the four foamixes are given in Table XX.

Distress Prediction

The prediction hypothesis for the subsystem assumes that the observed distress (cracking or rutting) is a function of the state of strain induced by traffic loads. For the purpose of this study, the damage is defined as the conditions of a single load application in contributing to cracking or rutting. The distress is the cumulative effect of damage resulting in an observable and measurable amount of cracking or rutting.

A cumulative damage hypothesis based on linear summation of cycle ratios was adopted in this study. The cumulative damage (distress) hypothesis states that the materials mechanistic responses, vertical compressive strain (ε_c) at the subgrade surface or tensile strain (ε_t) at the underside of the treated layer, will reach its limiting condition when

$$\sum_{i=1}^{i=n} \frac{n_i}{N_i} = \sum_{i=1}^{i=n} d_i = 1.0$$

where:

n_i = Number of 18,000-lb load repetitions applied during period, i;

TABLE XX

TENSILE STRAIN CRITERIA USED FOR ASPHALT TREATED LAYERS

Material Designation	Percent Asphalt Content	Percent Asphalt Volume V_b	Percent Air Voids V_v	Fatigue Cracking Model of Asphalt Pavement
AASHTO	5.4	11.0	5.0	$.07958 \times (\varepsilon_t)^{-3.291} \times (M_R*)^{-.854}$
Foamix (N-1)	5.5	9.30	22.0	$.0010 \times (\varepsilon_t)^{-3.291} \times (M_R^*)^{-.854}$
Foamix (B-1)	4.5	8.8	14.2	$.0026 \times (\varepsilon_t)^{-3.291} \times (M_R^*)^{-.854}$
Foamix (C-1)	4.0	9.0	5.0	$.0470 \times (\varepsilon_t)^{-3.291} \times (M_R^*)^{-.854}$
Foamix (C-2)	4.5	9.0	8.6	$.0110 \times (\varepsilon_t)^{-3.291} \times (M_R^*)^{-.854}$

*All M_R values change monthly during the year.

N_i = Total allowable 18,000 lb load repetitions for strain value, ε_c or ε_t, calculated during period i, as obtained from either Equation (6.1) or Equation (6.5);

d_i = Damage during period (i) and

n = Number of periods.

Method of Foamixes Structural Evaluation

The purpose of the study was to develop thickness equivalency factors for the four foamixes; (N-1), (B-1), (C-1) and (C-2) relative to the asphalt concrete used in the AASHTO Road Test surface course. The pavement structure analyzed was a full depth two layered system similar to that shown in Figure 3(a) (Chapter II). As applied to this study, the thickness equivalency factor was defined as the quotient obtained by dividing a foamix thickness T1 by an AASHTO asphalt concrete pavement thickness T where T1 and T would reach a certain level of distress, as predicted by models in Equation (6.1) or (6.5), when carrying the same number of equivalent 18,000 lb single axle load repetitions during the same period and under the same environmental condition when both pavements were constructed on the same subgrade.

All analyses were adjusted to reflect Oklahoma weather and environmental conditions. Table XXI presents the mean monthly air temperature for the last 30 years determined by averaging temperatures from 91 recording stations spread throughout the state of Oklahoma. The resilient moduli of the foamixes at different temperatures were determined in the laboratory using a diametral resilient modulus apparatus. The relation between resilient moduli of foamixes (N-1), (B-1), (C-1) and (C-2) [at their design asphalt contents] and temperature is illustrated in

TABLE XXI

ESTIMATED MEAN MONTHLY AIR TEMPERATURE (^{o}F) FOR OKLAHOMA

January	February	March	April	May	June	July	August	September	October	November	December
38.0	42.5	49.2	61.2	69.0	78.0	82.0	81.5	74.0	63.0	50.0	41.0

Figure 20. The resilient modulus of the AASHTO Road Test surface course as a function of temperature was determined by Finn et al. (53) and is also shown in Figure 20. Three different levels of subgrade resilient modulus at normal conditions and at a deviator stress of 6.0 psi [recommended by the Asphalt Institute (49)] were; 4,500, 12,000 and 22,500 psi. To represent the freeze and thaw effects on the subgrade, a subgrade resilient modulus of 50,000 psi was assigned to the freezing period (49, 52). Throughout one year; the time of freeze (ΔT_F) time of critical thaw (ΔT_C), time of thaw recovery (ΔT_R), time of normal subgrade condition (ΔT_N) and the thaw reduction factor (R_T) representing the South Carolina environmental conditions (49), were assumed to be the same for Oklahoma with the exception that the freeze was assumed to start in December rather than in January. Conditions used to represent the Oklahoma freeze and thaw effects for each level of normal subgrade modulus are shown in Table XXII and the subgrade moduli throughout one year for the three levels of normal subgrade moduli are shown in Table XXIII.

The thickness equivalency factors for the four foamixes were determined, using the DAMA computer program. For each of the four foamixes and the reference asphalt concrete (AASHTO Road Test surface course) 6 curves were developed. Two curves were developed at each of the three levels of subgrade resilient moduli, i.e., 4,500, 12,000 and 22,500 (normal condition). One of the curves related thickness of treated pavement to the total number of equivalent 18,000 lb single axle load repetitions (EAL) based on the vertical compressive strain (ε_c) on top of the subgrade using the model given in Equation (6.1). The other curve represented the same relation, EAL versus pavement thickness, based on the horizontal tensile strain (ε_t) at the underside of the asphalt

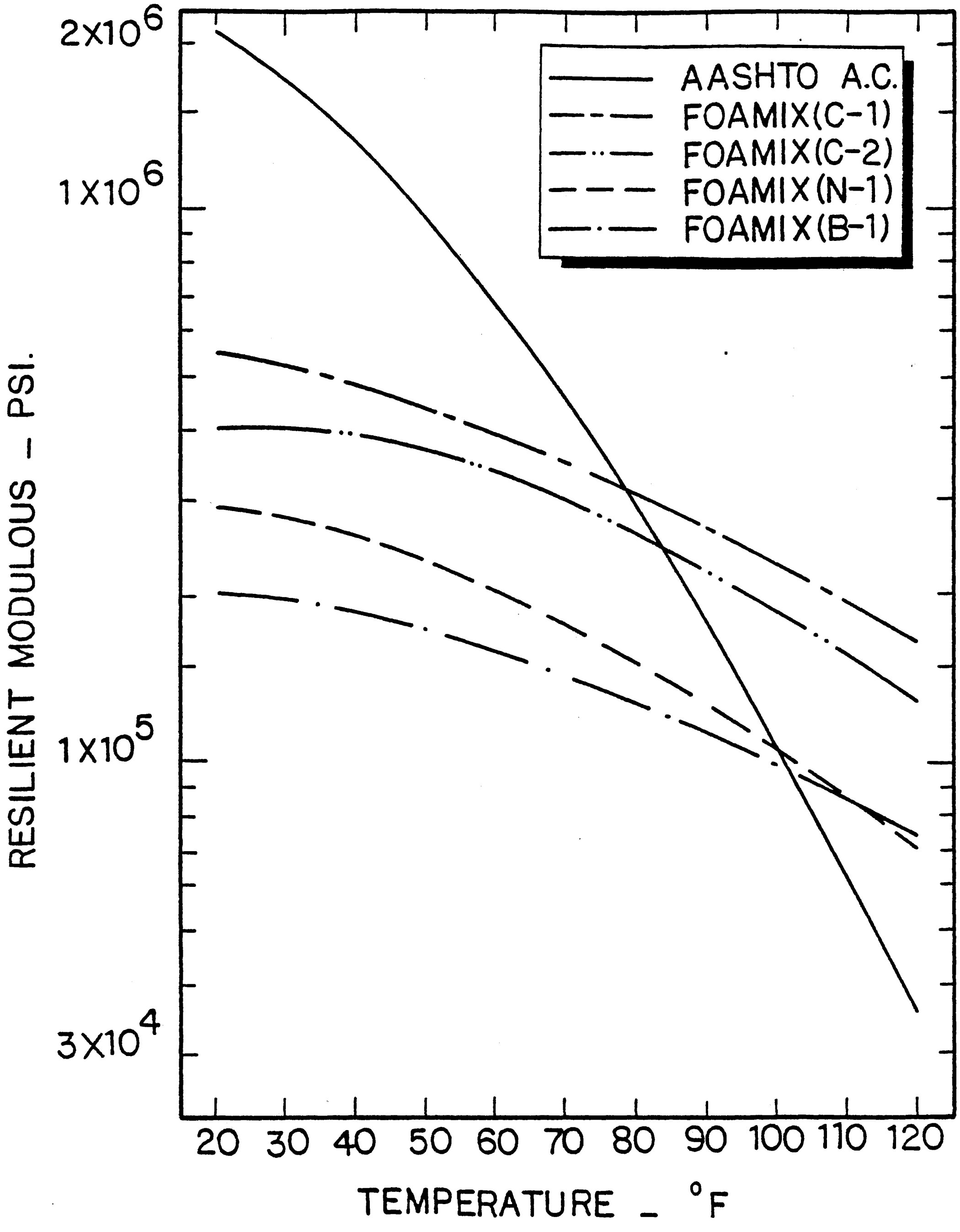


Figure 20. Resilient Modulus Vs. Temperature Relationship

TABLE XXII

CONDITIONS USED TO REPRESENT FREEZE AND THAW EFFECTS ON OKLAHOMA SUBGRADES

$M_R(N)$, PSI	$M_R(F)$ PSI	R_T	$M_R(T)$, PSI ($M_R(N) x R_T$)	T_{F_0}	ΔT_F, Month	ΔT_C, Month	ΔT_R, Month	ΔT_N, Month
4,500	50,000	0.3	1,350	December	2	1	4	5
12,000	50,000	0.6	7,200	December	2	1	4	5
22,500	50,000	0.8	18,000	December	2	1	4	5

TABLE XXIII

MONTHLY RESILIENT MODULI FOR OKLAHOMA SUBGRADES

	Time of Freeze		Critical Thaw	Time of Thaw Recovery				Time of Normal Subgrade Condition				
	Subgrade Modulus by Month, PSI											
$M_R(N)$ PSI	JAN.	FEB.	MAR.	APR.	MAY	JUN.	JUL.	AUG.	SEPT.	OCT.	NOV.	DEC.
4,500	27,300	50,000	1,350	2,140	2,930	3,710	4,500	4,500	4,500	4,500	4,500	4,500
12,000	31,000	50,000	7,200	8,400	9,600	10,800	12,000	12,000	12,000	12,000	12,000	12,000
22,500	36,300	50,000	18,000	19,100	20,300	21,400	22,500	22,500	22,500	22,500	22,500	22,500

treated layer using the model given in Equation (6.5).

For an assumed pavement thickness, ranging from 4 to 40 in., the monthly damage, d_i, was computed throughout the year assuming only one repetition per month. The yearly damage (D_{years}) equals the sum of monthly damages (Σd_i), and represents the total damage predicted after one year as a result of 12 load repetitions equally distributed (on monthly basis) throughout the year, or;

$$D_{year} = \sum_{i=1}^{1=12} di$$

where:

D_{year} = Yearly damage due to 12 repetitions;

d_i = Monthly damage due to 1 repetition every month.

Knowing D_{year}, which has the advantage of being a damage coefficient representing all environmental changes during the year, the total number of repetitions to reach distress, N_f, was calculated as:

$$N_f = \frac{12}{D_{year}}$$

From the pavement thickness and the corresponding N_f values the curves relating pavement thickness and number of repetitions to reach a damage coefficient equal to 1.0 were drawn. Each curve represented one of the five materials at one of the three levels of subgrade resilient modulus using one of the two design criteria [compressive strain on top of the subgrade (ε_c) or tensile strain in the underside of the treated layer (ε_t)]. Thirty curves were developed for the above mentioned combinations.

The thickness equivalency factor, as defined previously, was then calculated from information obtained from the 30 developed curves. For a given design total number of repetitions (EAL), the AASHTO full depth asphalt concrete thickness (T) was found. This thickness was considered the reference thickness for the four foamixes studied. The design thickness (T1), for any of the four foamixes could then be found from the ε_t or ε_c curves. The thickness equivalency factor for a given foamix pavement designed for a damage coefficient of 1.0 is the quotient obtained by dividing T1 by T, or;

$$\text{Thickness Equivalency Factor} = \text{E.F.} = \frac{T1}{T} \qquad (6.6)$$

The above described procedure is shown in a schematic form in Figure 21.

Results and Discussion of Analytical Study

Based on the Oklahoma environmental conditions, a thickness design chart for a full depth AASHTO asphalt concrete was developed and is shown in Figure 22. The Asphalt Institute design charts for full depth asphalt concrete, recently published in the Manual Series MS-1 (55) were also used to determine design thicknesses for the AASHTO asphalt concrete layers. Table XXIV presents some of the design thicknesses, obtained by the respective methods, for comparison. The differences, in most cases, are considered small; however, in designing foamixes to replace portions of full depth asphalt concrete pavements, it is desirable to use the chart developed for Oklahoma environmental conditions. The Asphalt Institute charts were developed based on average conditions nationwide and may not be entirely suitable for use in Oklahoma.

Some of the thickness equivalency factors for foamixes (C-1), (C-2), (B-1) and (N-1) are given in a tabular form for the three subgrade modulus levels in Tables XXV, XXVI, XXVII and XXVIII, respectively.

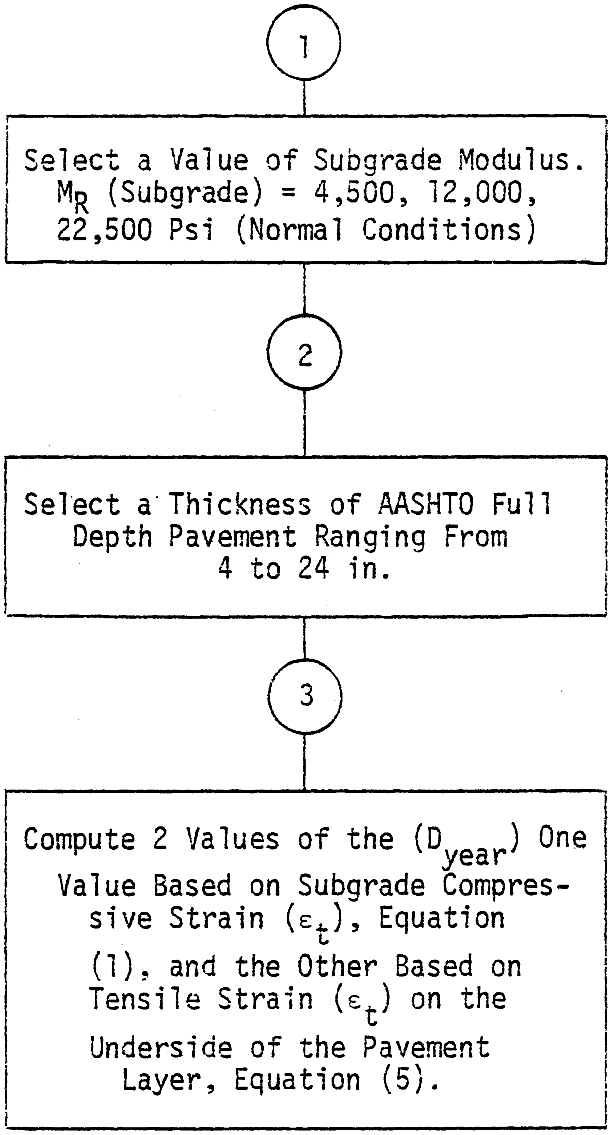


Figure 21. Scheme for Computing Thickness Equivalency Factors for the Foamixes

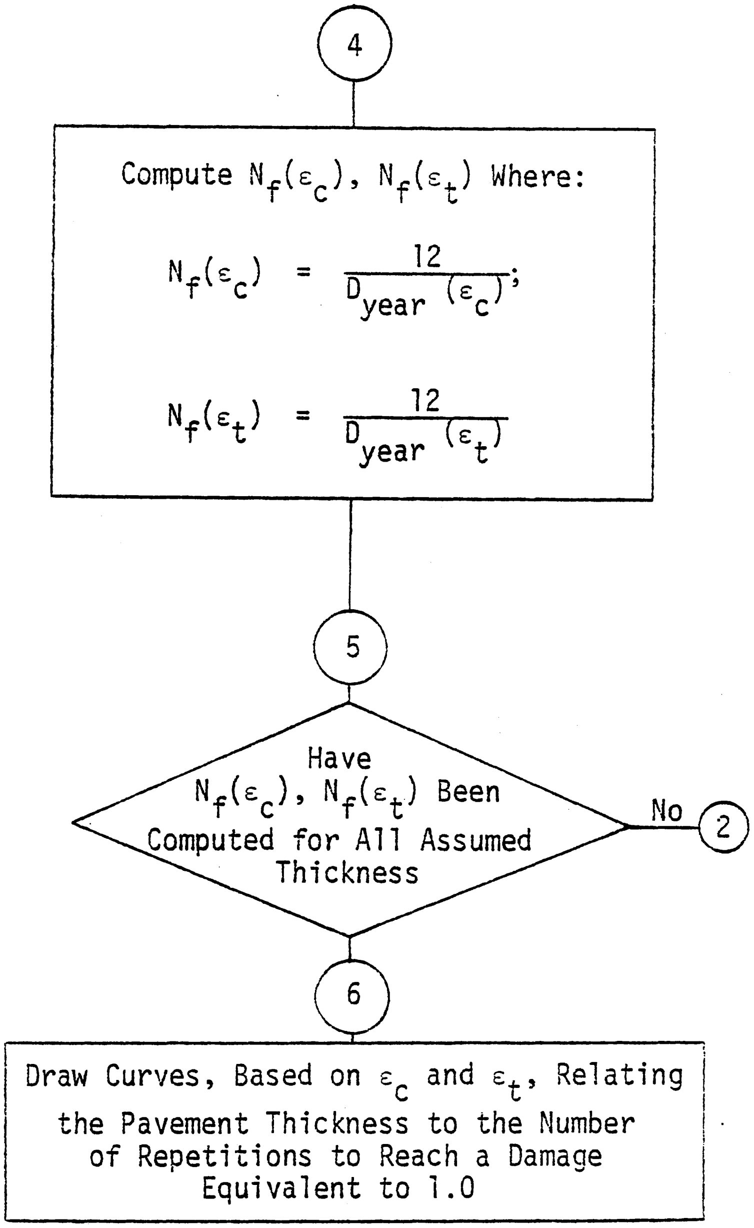
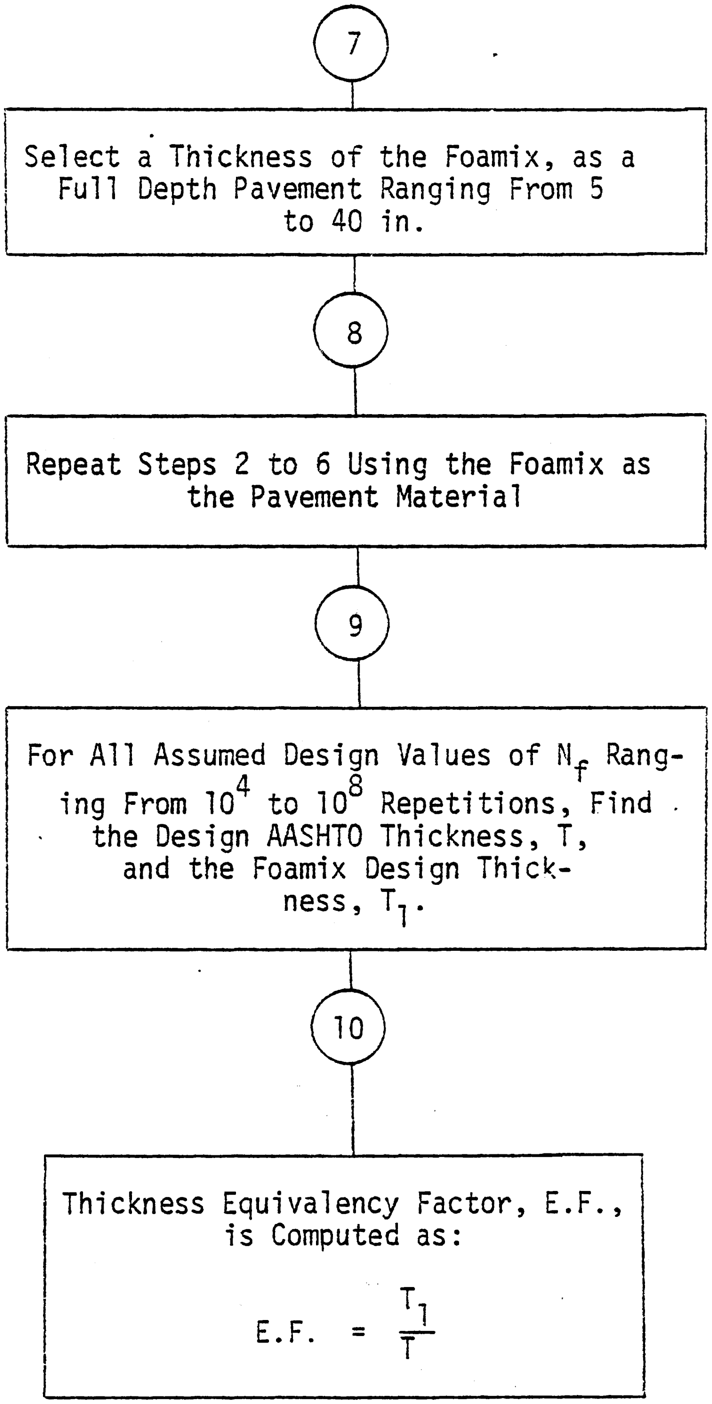


Figure 21 (Continued)



Select a Thickness of the Foamix, as a Full Depth Pavement Ranging From 5 to 40 in.

Repeat Steps 2 to 6 Using the Foamix as the Pavement Material

For All Assumed Design Values of N_f Ranging From 10^4 to 10^8 Repetitions, Find the Design AASHTO Thickness, T, and the Foamix Design Thickness, T_1.

Thickness Equivalency Factor, E.F., is Computed as:

$$E.F. = \frac{T_1}{T}$$

Figure 21 (Continued)

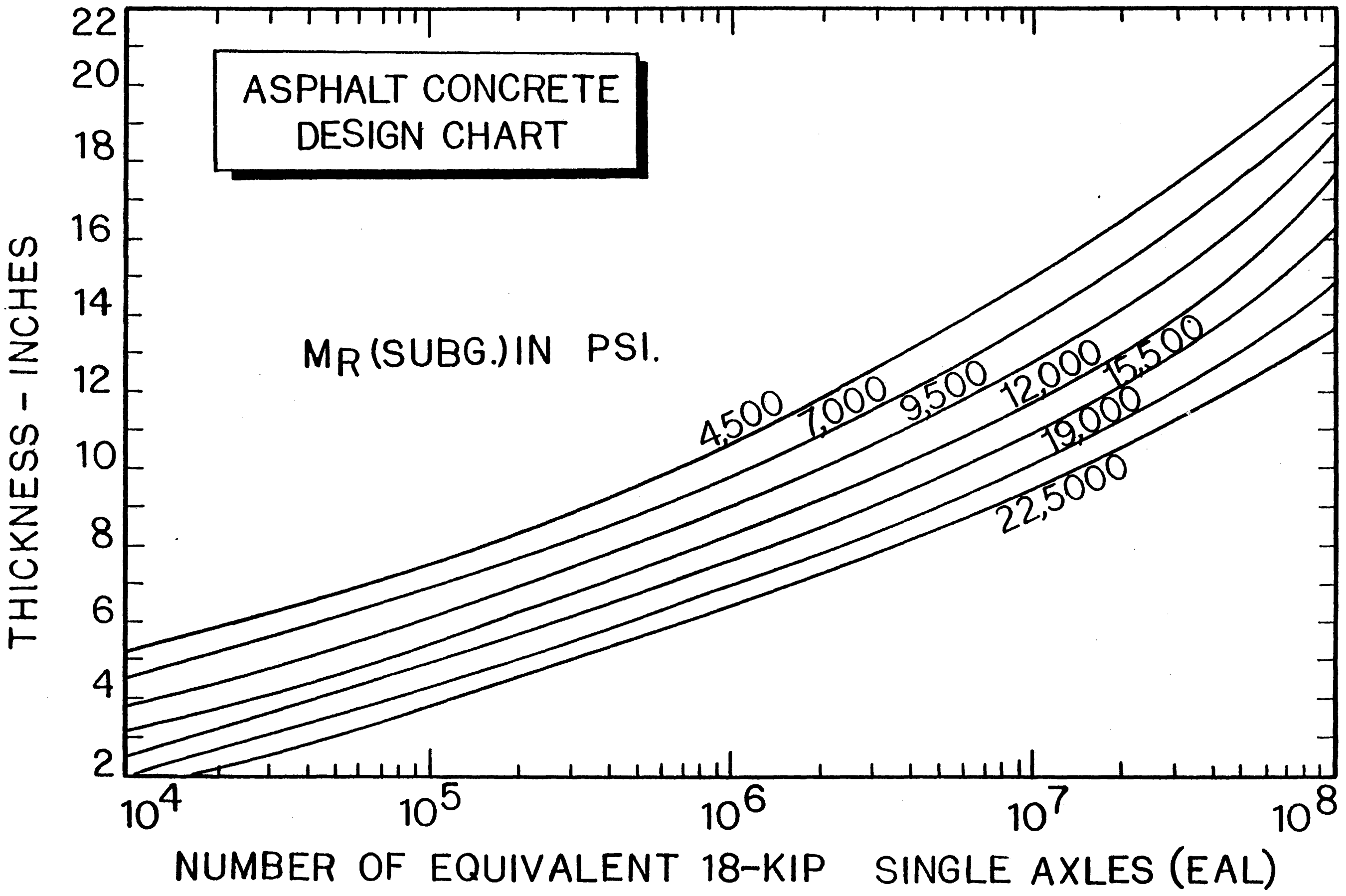


Figure 22. Full Depth AASHTO Asphalt Concrete Design Chart Developed for Oklahoma Environmental Conditions

TABLE XXIV

COMPARISON OF ASPHALT CONCRETE THICKNESS USING THE OKLAHOMA-AASHTO AND THE ASPHALT INSTITUTE CHARTS

Subgrade M_R, PSI	Number of Repetition, EAL	Design Thickness, in.		Difference in Thickness, in.
		OKLASHTO	Asphalt Inst.	
4,500	10^5	7.50	7.0	+0.5
	10^6	10.50	10.0	+0.5
	10^7	15.0	15.0	0.0
12,000	10^5	5.3	5.0	+0.3
	10^6	8.2	8.0	+0.2
	10^7	11.8	13.0	+1.2
22,500	10^5	3.8	4.0	0.2
	10^6	6.4	6.4	0.0
	10^7	9.5	11.1	+1.6

TABLE XXV

DESIGN THICKNESSES AND THICKNESS EQUIVALENCY FACTORS FOR FOAMIX (C-1)

Subgrade M_R, PSI	EAL	AASHTO T_{ε_t}	FOAMIX $T1_{\varepsilon_t}$	AASHTO T_{ε_c}	FOAMIX $T1_{\varepsilon_c}$	E.F. ε_t	E.F. ε_c	Design E.F.
4,500	10^4	2.15	2.75	5.15	4.8	1.28	0.93	.93
	10^5	5.0	5.75	7.5	7.0	1.15	0.96	.96
	10^6	8.65	9.6	10.6	9.75	1.11	0.92	.92
	10^7	13.6	14.8	14.9	13.6	1.09	0.91	.99
	10^8	20.0	21.75	20.3	18.9	1.08	0.93	1.07
12,000	$3 \times 10^{4*}$	1.1	2.2	4.2	3.8	7.0	0.9	.9
	10^5	2.9	3.9	5.4	4.9	1.35	0.9	.91
	10^6	6.7	7.8	8.2	7.5	1.16	0.91	.95
	10^7	11.4	12.9	11.8	10.8	1.13	.92	1.093
	10^8	17.65	19.4	17.0	15.4	1.09	.91	1.10
22,500	10^5	1.2	2.3	3.8	3.5	1.91	.9	.92
	10^6	5.0	6.35	6.4	5.9	1.27	.92	.99
	10^7	9.7	11.25	9.6	8.9	1.16	.93	1.16
	10^8	15.8	17.85	14.2	13.2	1.13	.93	1.13

*An EAL value less than this will result in an impractical value of T_{ε_t}.

TABLE XXVI

DESIGN THICKNESSES AND THICKNESS EQUIVALENCY FACTORS FOR FOAMIX (C-2)

Subgrade M_R, PSI	EAL	AASHTO T_{ε_t}	FOAMIX $T1_{\varepsilon_t}$	AASHTO T_{ε_c}	FOAMIX $T1_{\varepsilon_c}$	E.F. ε_t	E.F. ε_c	Design E.F.
	10^4	2.15	4.4	5.15	5.25	2.05	1.02	1.02
	10^5	5.0	7.9	7.5	7.36	1.58	1.02	1.05
4,500	10^6	8.65	12.35	10.6	10.6	1.43	1.0	1.17
	10^7	13.6	18.75	14.9	14.7	1.38	.99	1.26
	10^8	20.0	28.2	20.3	19.3	1.41	.95	1.39
	$3 \times 10^{4*}$	1.10	2.75	4.2	4.4	2.5	1.05	1.05
	10^5	2.9	6.0	5.4	5.3	2.07	.98	1.11
12,000	10^6	6.7	10.5	8.2	7.9	1.57	.96	1.28
	10^7	11.4	16.5	11.80	11.8	1.47	1.0	1.4
	10^8	17.65	24.3	17.0	16.65	1.37	.98	1.37
	10^5	1.2	4.4	3.8	3.83	3.6	1.0	1.16
22,500	10^6	5.0	8.85	6.4	6.37	1.77	1.0	1.38
	10^7	9.7	14.7	9.6	9.5	1.52	.99	1.32
	10^8	15.8	22.25	14.2	14.1	1.41	.99	1.42

*An EAL value less than this will result in an impractical value of T_{ε_t}.

TABLE XXVII

DESIGN THICKNESSES AND THICKNESS EQUIVALENCY FACTORS FOR FOAMIX (B-1)

Subgrade M_R, PSI	EAL	AASHTO T_{ε_t}	FOAMIX $T1_{\varepsilon_t}$	AASHTO T_{ε_c}	FOAMIX $T1_{\varepsilon_c}$	E.F. ε_t	E.F. ε_c	Design E.F.
4,500	10^4	2.15	8.8	5.15	7.3	4.1	1.42	1.7
	10^5	5.0	13.9	7.6	10.25	2.78	1.35	1.83
	10^6	8.65	21.1	10.55	14.65	2.44	1.39	2.0
	10^7	13.6	29.8	14.9	20.4	2.19	1.37	2.0
12,000	$3 \times 10^{4*}$	1.1	8.55	4.2	5.9	7.77	1.4	2.04
	10^5	2.9	11.3	5.4	7.4	3.89	1.37	2.09
	10^6	6.7	18.1	8.2	10.9	2.7	1.33	2.21
	10^7	11.4	26.2	11.8	16.2	2.3	1.37	2.22
	10^8	17.8	35.90	1.7	22.35	2.0	1.31	2.02
22,500	10^5	1.2	9.1	3.8	5.4	7.6	1.42	2.4
	10^6	5.0	15.25	6.4	8.45	3.05	1.32	2.38
	10^7	9.65	23.35	9.65	12.9	2.42	1.34	2.42
	10^8	15.8	32.85	14.2	18.8	2.08	1.32	2.08

*An EAL value less than this will result in an impractical value of T_{ε_t}.

TABLE XXVIII

DESIGN THICKNESSES AND THICKNESS EQUIVALENCY FACTORS FOR FOAMIX (N-1)

Subgrade, M_R, PSI	EAL	AASHTO T_{ε_t}	FOAMIX $T1_{\varepsilon_t}$	AASHTO T_{ε_c}	FOAMIX $T1_{\varepsilon_c}$	E.F. ε_t	E.F. ε_c	Design E.F.
4,500	10^4	2.15	9.7	5.15	6.6	4.5	1.28	1.88
	10^5	5.0	1.5	7.6	9.4	3.0	1.25	1.97
	10^6	8.65	21.8	10.55	13.5	2.4	1.27	2.066
	10^7	13.6	31.3	14.9	18.75	2.3	1.26	2.11
12,000	$3 \times 10^{4*}$	1.1	9.4	4.2	5.5	8.5	1.31	2.23
	10^5	2.9	12.53	5.40	6.9	4.33	1.28	2.32
	10^6	6.7	19.45	8.2	10.1	2.93	1.23	2.37
	10^7	11.4	27.4	11.8	15.0	2.4	1.27	2.32
	10^8	17.8	36.0	1.7	20.8	1.75	1.22	2.02
22,500	10^5	1.2	10.4	3.8	5.0	8.7	1.32	2.73
	10^6	5.0	17.0	6.4	7.9	3.4	1.23	2.66
	10^7	9.65	25.1	9.55	11.85	2.6	1.23	2.6
	10^8	15.8	35.3	14.2	17.75	2.25	1.25	2.23

*An EAL value less than this will result in an impractical value of T_{ε_t}.

It is evident from these tables that the thickness equivalency factors for a given material are not constant and that they change depending on the thickness of the replaced asphalt concrete layer and on the subgrade resilient modulus. This fact indicates a fallacy in the use of a single value for the thickness equivalency of a given material, as has been common practice in the past.

Charts relating the thickness equivalency factors of the four foamixes to the AASHTO asphalt concrete full depth thickness are shown in Figures 23, 24, 25, and 26. Foamix (C-1), a foamixture consisting of 4.0 percent by dry weight foamed asphalt and lime stone screenings has an excellent structural potential, i.e., a thickness equivalency varying from 0.9 to 1.16. A foamix of 4.5 percent by dry weight foamed asphalt and the crushed sand, foamix (C-2), has a thickness equivalency factor ranging from 1.02 to 1.52. This range of thickness equivalency is also considered very good for this relatively coarse crushed sand, at this low asphalt content. Foamix (N-1), very fine sand foamixed with 5.5 percent asphalt, shows a relatively poor structural ability. The thickness equivalency factors for this material were between 1.8 and 2.7. A slightly better structural potential is noticed for foamix (B-1) [blended sand mixed with 4.5 percent foamed asphalt]. As seen in Figure 26, the thickness equivalency factor range from 1.6 to 2.4. This improvement is mainly due to the lower air voids content, 14.2 percent for foamix (B-1) compared to 22.0 percent for foamix (N-1). The higher thickness equivalency values for foamixes (B-1) and (N-1) is due to their lower fatigue resistance. In most cases, fatigue cracking as represented by ε_t controlled their design thicknesses. If the thickness equivalency factors computed for these foamixes; (N-1) and (B-1), were based only on the

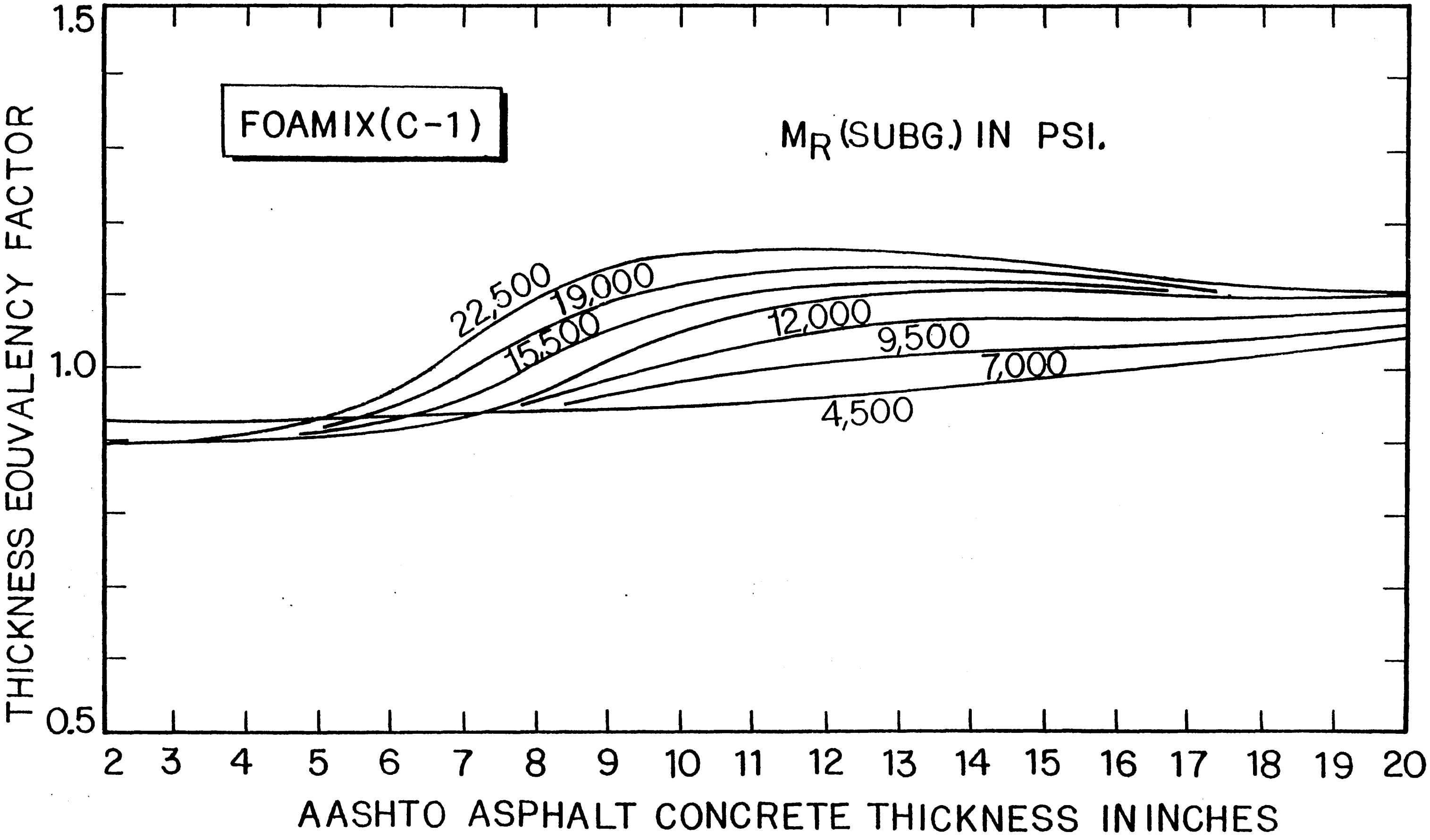


Figure 23. Design Thickness Equivalency Factors Chart for Foamix (C-1), Screenings

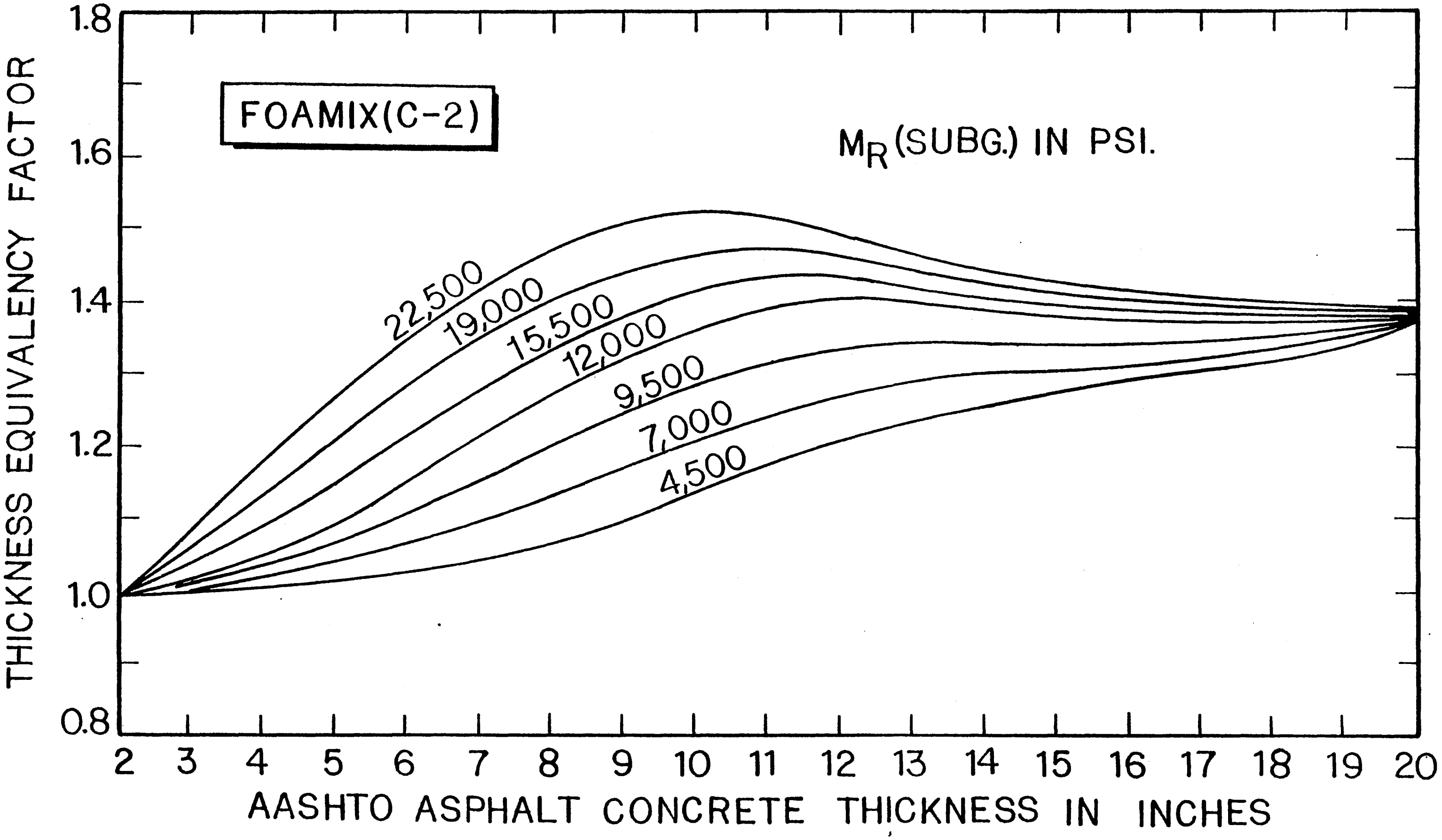


Figure 24. Design Thickness Equivalency Factors Chart for Foamix (C-2); Crushed Sand

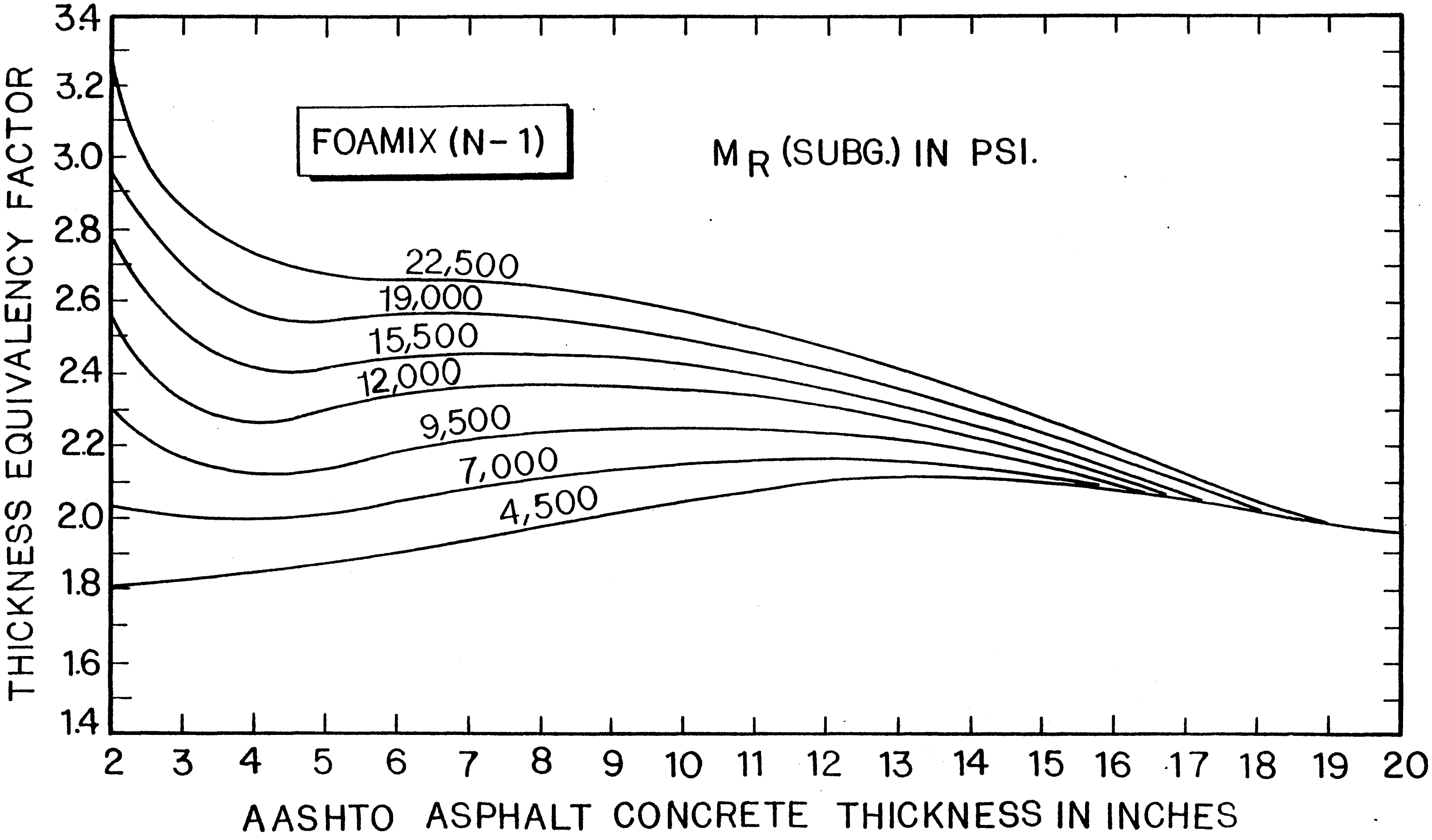


Figure 25. Design Thickness Equivalency Factors Chart for Foamix (N-1); Yukon Sand

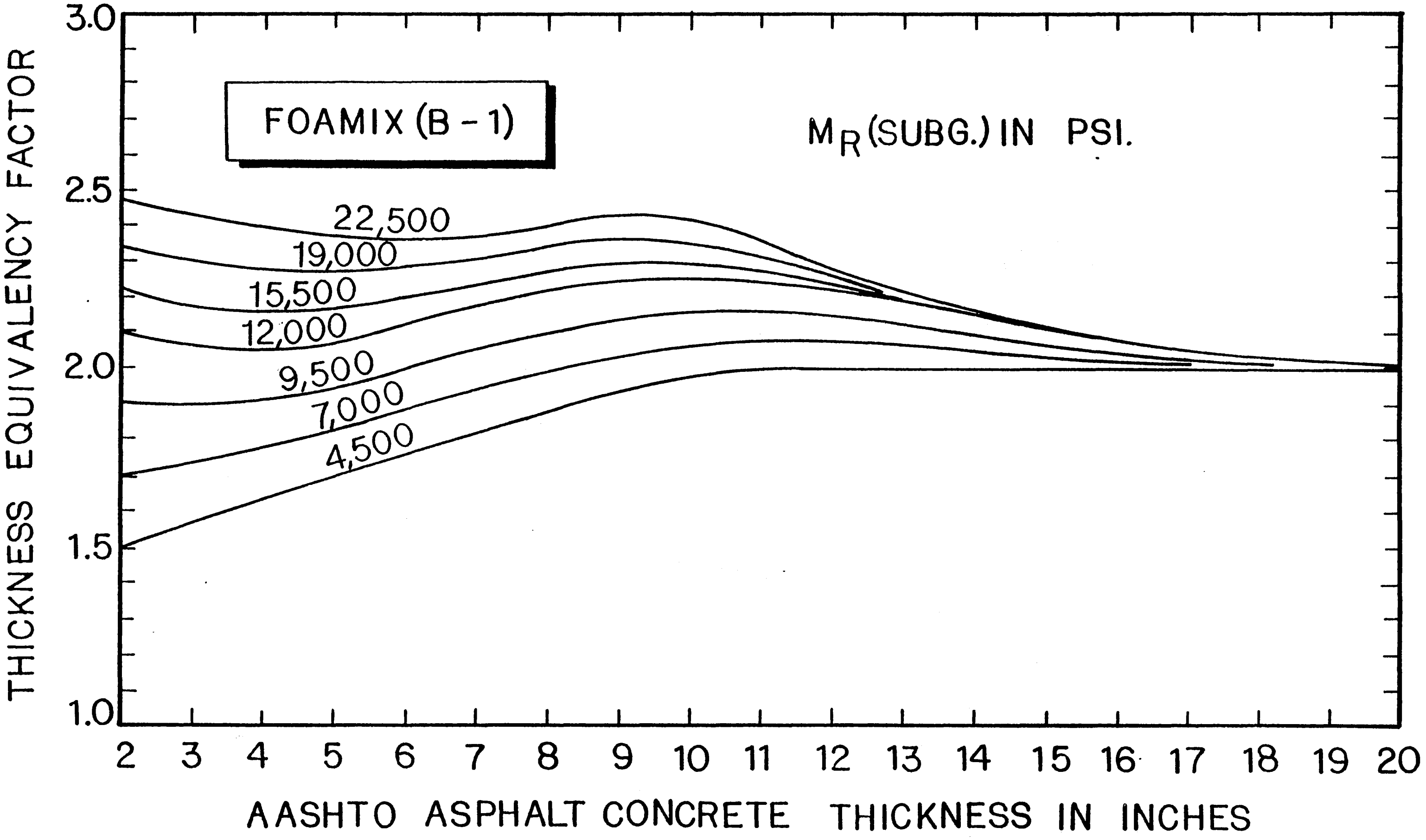


Figure 26. Design Thickness Equivalency Factors Chart for Foamix (B-1); Blended Sand

compressive strain (ε_c) on top of the subgrade layer the values would be much lower. Tables XXVII and XXVIII shows that the thickness equivalencies based on (ε_c) range from 1.31 to 1.42 for foamix (B-1) and from 1.22 to 1.32 for foamix (N-1). Considerable advantage could be gained by using these equivalencies instead of those based on ε_t or the design equivalencies determined as previously described. Some indication of a precedent for doing this is found in the Chevron pavement design procecure (52) page 431, where Santucci stated the following:

> "It is our experience that mixes with extremely high void contents (> 20%), seldom fail in the field by fatigue. It is conceivable that the primary thickness design consideration for these mixes is vertical subgrade strain. Permanent deformation of the mix itself is also an important design consideration for such mixes."

However, as suggested by the statement, Santucci's concern with the permanent deformation of mixes with high air void contents is evident and will probably govern the actual design.

It is also noticed from Figures 23 through 26 that the thickness equivalency curves, for the four foamixes, tends to converge at an asphalt concrete thickness of about 20 inches. The thickness equivalency factors at an asphalt concrete thickness of 13.0 inches and above become relatively constant with values near 1.1, 1.4, 2.0 and 2.0 for foamixes (C-1), (C-2), (N-1) and (B-1), respectively. In Figures 25 and 26 the thickness equivalency curves tend to diverge at an asphalt concrete thickness of 6.0 inches and below. Also the diverging curves tend toward increasing thickness equivalency values at the higher subgrade moduli. An interpretation of this tendency is that; these two foamixes, (N-1) and (B-1), tend to act as untreated layers when they are constructed over a subgrade with a high resilient modulus. The subgrade, in this case, is strong enough not to allow layers of relatively low stiffness to act as a reinforcing layer.

CHAPTER VII

CONCLUSIONS AND RECOMMENDATIONS

Conclusions

Within the scope of this study and on the basis of materials studied the following conclusions were drawn:

1. Foamed asphalt is an effective binder for stabilizing and upgrading many marginal quality fine aggregates for use in road construction.

2. Proper laboratory mix design procedures, including acceptance criteria for aggregates and suitable test methods for foamixes, must be employed to achieve the full potential of foamed asphalt usage.

3. The "particle index" (a measure of angularity and surface texture of aggregate particles) is an excellent parameter or indicator for determining the suitability of marginal aggregates for foamixing.

4. Compacted foamixes made with aggregates having particle index values greater than 10 had significantly higher stability, shear resistance and resilient modulus values and were less susceptible to moisture intrusion.

5. The amount of fines (percent of material passing the No. 200 sieve) contained in a fine aggregate greatly influences the quality and properties of the foamix produced. A minimum of 4.0 percent fines is needed to improve distribution of the foamed asphalt during mixing and the resulting stability, shear resistance and resilient modulus of the compacted foamix.

6. The optimum premolding moisture content for a foamix is a function of aggregate's optimum moisture content, the percent of fines in the

aggregate and the percent asphalt content in the mix. This moisture content can be determined from a predictive equation without the necessity of extensive preliminary laboratory testing.

7. Initially, compacted foamixes have little resistance to deformation but gain strength rapidly as moisture is lost by evaporation. In field applications, strength gains beyond those at complete desiccation are expected due to the kneading action of traffic and stiffening of the binder.

8. The rate of curing, or rate of moisture loss by evaporation is dependent on the foamix's percent air voids, percent asphalt content and thickness in laboratory specimens as well as in a pavement structure.

9. The vane shear test can be used to evaluate the shear strength of compacted foamixes made with fine aggregates which have particle indicies less than 10. The test apparatus and procedure can be employed both in the laboratory and in field applications.

10. At asphalt levels of 4.0 percent and above the foam process may be effectively used with materials which have particle indices greater than 10 to provide excellent bases as attractive alternatives to conventional hot asphalt mixes. Based on the use of mathematical models, it was found that from 0.9 to 1.5 inches of these foamix bases is equivalent to 1.0 inch of the conventional asphalt concrete used in the AASHTO Road Test surface course.

Recommendations

In view of the observations and conclusions made in this investigation, the following recommendations are presented:

1. Additional research is needed on a wider range of Oklahoma

aggregates to develop a more general model relating the optimum mixing and molding moisture contents to the aggregate properties. Percent material passing sieve No. 200, optimum moisture content of the aggregate and the percent foamed asphalt contents should be considered as the primary independent variables.

2. The particle index test should be adopted as one of the basic evaluative procedures for the suitability of an aggregate for foamixing. This test value together with a specified percent fines of at least 4.0 percent are believed to be the principal factors in predicting the strength of the evaluated foamix.

3. The Asphalt Institute moisture exposure procedure (19) is recommended for evaluating the moisture susceptibility of foamixes made with aggregates of particle indices greater than 10. However, this procedure is considered too severe for materials having particle indices less than 10. Therefore, a new method of exposing compacted specimens to moisture intrusion is needed.

4. Verification of the fatigue properties of the four foamixes studied can be achieved by performing controlled stress beam fatigue tests. Comparison between thickness equivalencies as found in Chapter VI and those found employing laboratory determined fatigue curves would be useful in future studies of foamixes.

5. An economic study which includes a cost comparison of foamixes with other conventional base course materials should be considered.

6. Studies should be conducted relative to actual paving projects in which foamix bases are used. Correlations of field data with data obtained in the laboratory could be used to refine the proposed mix design criteria and procedure developed in this study.

Summary of the Proposed Mix Design Procedure

A tentative mix design procedure for foamixes was developed. This procedure is based on the experience gained during this study and what has been proposed by others. A summary of the procedure is given below.

Aggregate Quality

The following aggregates properties must be determined.

1. Percent material passing No. 200 sieve.
2. Particle index value.
3. Bulk specific gravity.
4. Optimum aggregate moisture content.

A minimum of 4.0 percent material passing the No. 200 sieve and a minimum particle index of 5.0 are recommended to ensure a reasonably good foamix properties.

Foamed Asphalt

The asphalt cement temperature and the percent water added for foaming should be adjusted so that a foamed asphalt having a foam ratio from 8.0 to 15.0 and a half life of at least 20.0 seconds is produced. If the asphalt cement being used has a low foaming potential, a counter anti-foaming agent should be added to improve its foaming qualities.

Specimen Preparation

Mixing. The optimum premolding moisture content can be determined using Equation (5.1). The optimum mixing moisture content is obtained by increasing the percent water mixed in the aggregate in increments and mixing with foamed asphalt. The degree of asphalt cement distribution is determined as follows: A small portion of each mix is spread on a filter paper, then cured in an oven for 30 minutes. The optimum mixing moisture content is that which produces the darkest mix free of uncombined asphalt particles.

If the difference between optimum premolding and mixing moisture contents is low, the optimum premolding moisture content is recommended for both mixing and molding. If mixing moisture content is significantly higher than the premolding moisture content, precompaction curing is advised. To do this, the mix is set aside and periodically stirred to allow evaporation of moisture. The mix weight is recurrently checked until it loses an amount of water equal to the difference between the mixing and premolding moisture contents. Compaction can then be performed.

The amount of asphalt cement that adheres to the mixer's blade and bowl can be determined as in Appendix B or by any other acceptable method. Adjustment for this loss of asphalt cement and aggregate should be made.

Molding. Specimens should be compacted at room temperature using the Gyratory-shear molding procedure recommended by the Oklahoma Department of Transportation (39). The total batch weight should be that necessary to produce a specimen 4.0 in. in diameter and (2.0 ± 0.1) in height. Batch weight is dependent on the type of aggregate and can differ significantly from one material to another.

Test Procedure

Depending on the type of aggregate used in the foamix, two testing procedures are recommended. The first covers foamixes made from aggregates with particle indices greater than 10. The second procedure is recommended if the aggregate used in a foamix has a particle index less than 10.

Test Procedure-1: The following test values are required for compacted test specimens subjected to long curing.

1. Hveem Resistance and Stability at 105°F (25°C).

2. Hveem Resistance and Stability before and after moisture exposure at ambient temperature, 78°F (25°C).

3. Percent moisture absorbed by dry weight of foamix.

Test Procedure-2: The following test values are required for compacted test specimens subjected to long curing.

1. Hveem Resistance at 105°F (25°C).

2. Hveem Resistance before and after moisture exposure at ambient temperature, 78°F (25°C).

3. Vane Shear Stress at 105°F (25°C).

4. Percent moisture absorbed by dry weight of foamix.

The design criteria for test Procedure-1 and test Procedure-2 are those for heavy and light volume traffic respectively. These criteria are given in Table XXVIII.

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APPENDIX A

FOAM DISPENSER OPERATIONAL INSTRUCTIONS AND PROCEDURE FOR ESTABLISHING FOAMED ASPHALT QUALITY CONTROL

Foam Dispenser Operational Instructions

The following are step-by-step instructions for the operation of the foamix asphalt dispenser used in this study. More detailed instructions and procedures for calibrating the system and flushing the asphalt lines can be found in Reference 7.

Preliminary Connections and Settings

1. Connect air and water lines to the dispenser. Open air and water inlet valves.

2. Adjust air pressure to the control solenoid, which activates the three-way asphalt valve, to 35.0 psig, and air pressure to the foam nozzle solenoid to 25.0 psig. This adjustment is made by releasing the indicated control lock rings on the dispenser's control panel.

3. Adjust water pressure regulator to 50.0 psi.

4. Turn all switches and controller dials to the "off" position. Plug in the dispenser's power cord to a standard three-prong 120 volt electrical outlet.

Water System

1. Place a container under nozzle to catch water, then open water solenoid with switch in test (down) position. Maintain flow until all air is purged from the system.

2. Set air-to-nozzle solenoid switch in test (down) position to add atomization air, then set water flow rate to the desired water to asphalt percentage by adjusting the rotameter to the desired setting, as found from Figure 9.

3. Return air and water switches to "off" position.

Heating System

1. Set asphalt tank thermostat to desired operating temperature.

2. Turn on dispenser's main power switch. Turn on tank heaters, both main and quick heat switches (pilot lights for these heater elements on the control panel will be on). Allow asphalt tank to heat for at least one hour.

3. After heating the asphalt cement in the tank, turn on asphalt line heat controller and set to a temperature of $150^{O}F$ ($65^{O}C$). Wait until the line temperature, as indicated by the line temperature gauge, has stabilized before increasing the temperature. An elapsed time of 10 to 15 minutes is required before subsequent temperature settings.

4. Increase the asphalt line temperature in increments of $25^{O}F$ ($14^{O}C$), following the procedure in step 3, until the desired temperature is reached.

5. After the asphalt in the system, i.e., in the tank and lines, has reached the desired operating temperature, switch pump on momentarily then off to verify that it is warm enough to operate properly.

6. If pump hesitates, readjust heat controllers as required, wait a few minutes and repeat pump switch verification until circulation can be maintained.

Foamed Asphalt Production

1. After all settings for air pressure, water flow, and asphalt temperature are as desired, turn timer switch on (up position).

2. Place the 3 solenoid switches for asphalt, water, and air in run (up) position and the ready light should glow. Set the top asphalt timer for 2.0 or more seconds then push the momentary start button to activate

a foam production cycle. The asphalt foam produced is discarded. This wasted foam cycle flushes the nozzle and refills the hot asphalt cement lines. At the conclusion of this waste cycle, all timers are automatically reset.

3. Set the asphalt timer to deliver the desired weight of asphalt for a batch using the calibration chart of Figure 10 to convert from weight in grams to time in seconds. Position a tared can or the mixer bowl, if an aggregate batch is to be mixed, and push momentary start button to initiate foam sequence.

Asphalt Time Setting Calculations

The chart shown in Figure 10 relates various asphalt batch weights (in grams) to the corresponding time settings as indicated by the dispenser's asphalt timer. This chart was developed for an asphalt cement that had a specific gravity of 1.036 at 60°F (15.6°C). To use this chart for determining the time setting (in seconds) for an asphalt cement other than that used in developing this chart, a correction factor (C.F.) is calculated as follows:

$$\text{C.F.} = \frac{1.036}{\text{Specific Gravity of New Asphalt at } 60^{o}\text{F } (15.6^{o}\text{C})}$$

For an asphalt batch weight W, the corrected weight (W_c) used to find the corresponding time setting from Figure 10 is

$$W_c = W \times (\text{C.F.})$$

Example Calculations

The specific gravity of the asphalt cement used in this study was

0.994 at 77°F (25°C) (see Table VII). Determine the asphalt time setting for the dispenser's asphalt timer that is required to produce a delivered foamed asphalt batch weight W = 120.0 g.

1. Determine the specific gravity of the asphalt cement at 60°F (15.6°C) by dividing the specific gravity at 77°F (25°C) by the volume multiplier for correcting asphalt volume at 77°F (25°C) to the basis of 60° (15.6°C). From the temperature volume corrections in Table IV(1a) of Reference (56), the multiplier M_{77} = 0.9941, and

$$S.G._{60^oF} = \frac{S.G._{77^oF}}{.9941} \times \frac{\gamma_{w77^oF}}{\gamma_{w60^oF}} = 0.9982$$

2. Calculate the C.F. as defined previously

$$C.F. = \frac{1.036}{.9982} = 1.038$$

3. Compute the corrected weight W_c where:

$$W_c = W \times (C.F.)$$
$$= 120 \times (1.038) = 124.6 \text{ g.}$$

4. From Figure 10, find the time setting for (W_c = 124.6 g), or time setting = 3.5 seconds.

Procedure for Establishing Control of Foamed Asphalt Quality

The following procedure was used to establish the asphalt cement temperature, the amount of added water, and the amount of counter anti-foaming agent needed to achieve optimum quality of the delivered foamed asphalt as measured by the "Foam Ratio" and "Half Life". One gallon size empty food cans were used to receive and measure the volume of foamed asphalt batches. A dip-stick slightly longer than the vertical depth of these cans was volumetrically calibrated by marking its length in approximately 200 ml increments. The dip-stick calibration was accomplished by adding 200 g amounts of water to a can, holding the stick vertically so that the bottom touched the water surface, and then marking the stick at the upper rim of the can for each 200 g amount of water, until the can was full.

Procedure

1. Ready the foam dispenser (following operating instructions described in previous section) with an initial asphalt temperature of 325°F (163°C) and a water rotameter setting of 4.8 obtained from Figure 9. This setting will deliver 1.5 percent water by weight of asphalt delivered to the nozzle.

2. Set the asphalt delivery time controller to produce a batch of foamed asphalt weighing approximately 120.0 g using the calibration chart in Figure 10. The required time setting is 3.5 seconds.

3. Produce and discard one batch of foamed asphalt to ensure proper operation of the foam delivery system and to flush the foam nozzle.

4. Place a one gallon can of known weight, W_1, beneath the foam nozzle, then push the momentary start button for foam delivery. Start a stop watch immediately after completion of the foam cycle.

5. Measure the foam volume, immediately after it is injected into the can, using the calibrated dip-stick. An average height of the uneven

foam surface is estimated, the bottom of the dip-stick is lowered to this estimated point, and the volume indicated by the calibration mark at the can's rim is recorded as, V_f, in ml.

6. Make repeated volume measurements with the dip-stick as the foam dissipates. When the volume of the foam has subsided to one-half its initial volume, stop the watch and record the elapsed time, t, in seconds.

7. Weigh the can containing the foamed asphalt batch and record the weight as, W_2.

8. Steps 1 through 7 are repeated for each of the desired combinations of asphalt cement temperature and water flow rate. In this study six combinations were used; two temperatures, 325^oF (163^oC) and 350^oF (177^oC) and three water flow rates, 1.5, 2.0, and 2.5 percent of the added asphalt weight.

<u>Note</u>: With the asphalt used in this study it was necessary to add 0.70 percent (by weight of asphalt) of magnesium stearate, counter anti-foaming agent, to the asphalt in order to obtain the proper foaming characteristics.

Calculations

1. Determine the volume of asphalt batch before foaming, V_b, in ml.

$$V_b = \frac{W_2 - W_1}{G_b}$$

where:

W_1 = Tare weight of can, g.

W_2 = Weight of can and foamed asphalt batch, g.

G_b = Specific gravity of asphalt cement.

Note: The amount of water injected in hot asphalt during the foaming process is negligible since it evaporates upon the discharge of the foamed asphalt batch.

2. Determine foam quality parameters as follows:

$$\text{Foam Ratio} = \frac{V_f}{V_b}$$

$$\text{Half Life} = t \text{ seconds}$$

where:

V_f = Volume of foamed asphalt immediately after delivery, ml.

t = Elapsed time, in seconds, from completion of foam discharge to the moment the foam shrinks to one half its initial volume.

APPENDIX B

TRIAL MIXTURES AND CURING PROCEDURES CALCULATIONS

Trial Mixtures Calculations

The purpose of the trial mixtures, described in Chapter IV, was (1) to determine the amount of foamed asphalt required to be added to a batch to replace the amount of asphalt cement adhering to the mixer blade and bowl after mixing and (2) to determine the dry aggregate batch weight required to produce gyratory-shear compacted foamed asphalt-fine aggregate specimens 4.0 in. diameter by 2.0 in. height.

The following formulas were used in calculating the above mentioned values:

1. The amount of adherent foamed asphalt and the amount required to compute a corrected asphalt batch weight:

$$W_{bl} = C(\Delta W), \text{ and}$$

$$W_{bt} = W_{bc} + W_{bl}$$

where:

ΔW = Total weight of adhering material (aggregate and asphalt, on mixer's blade and bowl as found, for batches at different levels of foamed asphalt content, from procedure described in Chapter IV.

C = A factor equivalent to the ratio of the weight of adherent asphalt cement to the total weight of material adhering to the blade and bowl. (In this study, C was found to be approximately 1/3 for the range of asphalt content used in the batches.)

W_{bl} = Weight of asphalt adhering to the mixer's blade and bowl for a given weight of delivered foamed asphalt.

W_{bc} = Weight of foamed asphalt added to a batch; as computed on the basis of the dry weight of the aggregate batch, required to produce a mix at a specific asphalt content.

W_{bt} = Corrected foamed asphalt batch weight used in determining the time in seconds, using Figure 10, to be set on the dispenser's asphalt timer. The amount of delivered foamed asphalt will produce a foamed asphalt-mixture containing a weight of asphalt equal to W_{bc}.

2. Dry aggregate batch weight required to produce a set of three molded specimens with a 4.0 in. (102.0 mm) diameter and a 2.0 in (51.0 mm) height.

$$W_{batch} = \frac{2.0}{Ht} \cdot \frac{3W_{spec.}}{(1 + P_{ba} - P_{bl})} + \frac{2}{3}\,\Delta W \tag{1}$$

where:

W_{batch} = Weight of dry aggregate required to produce a batch of foamed asphalt-aggregate mixture which will make 3 molded specimens.

$W_{spec.}$ = Average dry weight of the compacted specimens produced using the procedure in Chapter IV (see Trial Mixtures).

H_t = Average height ofthe same compacted specimens (see $W_{spec.}$ above).

P_{ba} = Percent of asphalt sprayed from the nozzle, expressed as a decimal fraction on a dry weight basis, i.e., $\frac{W_{bt}}{3000}$ where 3000 g is the dry weight of aggregate in a batch. (See Trial Mixture procedure, Chapter IV.)

P_{bl} = Percent asphalt adhering to blade and bowl, expressed as a decimal fraction on a dry weight basis, equal to $\frac{W_{bl}}{3000}$.

Curing Procedures Calculations

Foamix specimens were tested or evaluated after three different periods of curing, i.e., short curing, intermediate curing and long curing. The procedure adopted for curing the specimens was discussed in Chapter IV (see Curing Procedures). For each of the curing conditions the "percent curing" is defined as the percent moisture lost during the curing period, by weight of the original molding moisture content. The percent curing values are computed as follows:

$$PSC = \left(\frac{W_t - W_s}{W_t - W_m}\right) \frac{(100 + w_{mold} + P_{bc})}{w_{mold}} \times 100 \qquad (B.2)$$

$$PIC = \left[\frac{(W_t - W_m) - W_i}{(W_t - W_m)}\right] \frac{(100 + w_{mold} + P_{bc})}{w_{mold}} \times 100 \qquad (B.3)$$

$$PLC = \left[\frac{(W_t - W_m) - W_l}{(W_t - W_m)}\right] \frac{(100 + w_{mold} + P_{bc})}{w_{mold}} \times 100 \qquad (B.4)$$

where:

PSC = Percent short curing.

PIC = Percent intermediate curing.

PLC = Percent long curing.

w_{mold} = Percent moisture content of mix at molding, by dry weight of aggregate.

P_{bc} = Percent asphalt cement content, by dry weight of aggregate.

W_m = Weight of PVC mold, g.

W_t = Weight of molded specimen, before any curing, plus weight of mold (W_m), g.

W_s = Weight of short cured specimen plus weight of mold (W_m), g.

W_i = Weight of specimen after intermediate curing, g.

W_l = Weight of specimen after long curing, g.

APPENDIX C

RESILIENT MODULUS EQUIPMENT DETAILS AND CALIBRATION

Test Equipment

The resilient modulus apparatus was fabricated in the laboratories of the OSU School of Civil Engineering. This equipment consists of a repetitive loading system, load and deformation sensors, a recorder, and test specimen positioning devices.

Repetitive Loading System

The air-actuated loading system applies a 75 lb (335.0 N) pulsating load of 0.10 second duration every 3.0 seconds across the diameter of a molded test specimen. This equipment is shown in the photograph of Figure 27 and includes the following components:

1. Compressed air system--a 6.0 gallon air tank, filter, pressure regulator, pressure gage and electrically timed solenoid valve connected in series to supply pulses of air to a pneumatic piston. The system was supplied with compressed air at 130 psig from a laboratory source.

2. Pneumatic piston--a Flaire Line pneumatic piston with a 2.5 in (63.0 mm) diameter and a stroke length of 3.0 in (76.0 mm) mounted in a reaction frame. The piston is activated by air pressure supplied from the solenoid valve and applies a pulsating load of specified duration and frequency.

3. Electrical controller--a Minarik Electric Company programmable micro-processor control unit, Model WP 6000, used to control the operation of the solenoid valve. The controller was programmed to open the valve for a 0.10 second interval at a 3.0 second frequency.

Load and Deformation Sensors

1. Load cell--a 300 lb (1340 N) capacity strain gage load cell

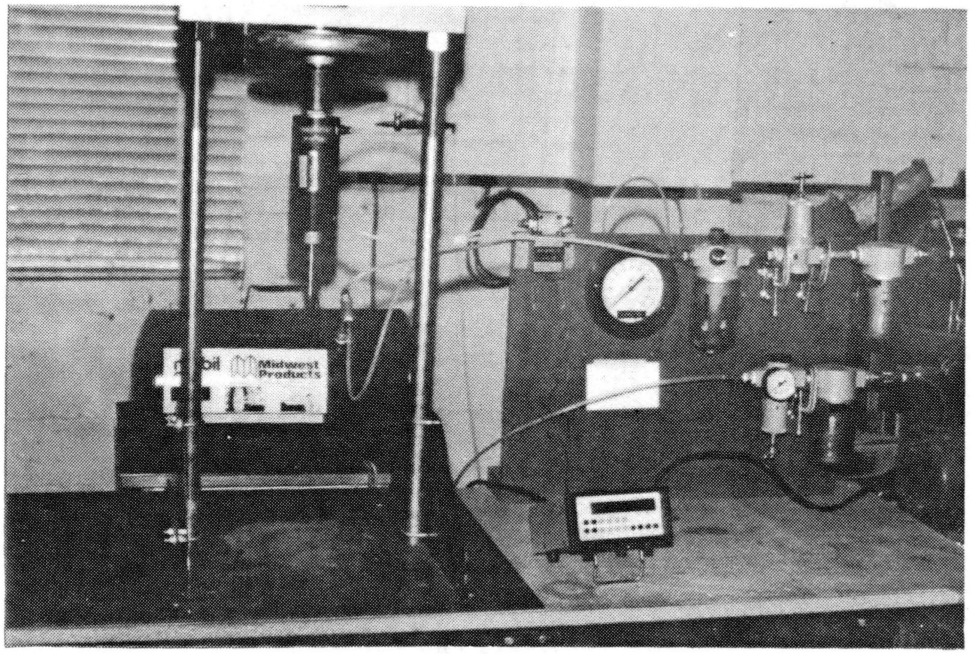


Figure 27. Repetitive Loading System (Compressed Air System, Pneumatic Piston and Load Function Generator)

placed between the ram of the pneumatic piston and a loading strip on top of the molded test specimen. The load cell senses the pulsating load as it is transmitted to the specimen and the output is recorded on a strip chart recorder. The positions of the piston, load cell and test specimen can be seen in Figure 11 (Chapter IV).

2. Displacement transducers--two Gould Statham universal transducers, Model UC-3, positioned horizontally and diametrically opposite on each side of a test specimen. The transducers are mounted in a yoke that is clamped to the specimen with the tips of the transducers barely touching the sides of the specimen (see Figure 28). Horizontal deformation of the specimen creates corresponding output voltages from the two transducers which are connected in parallel. This output voltage which represents the sum of deformations on both sides of the specimen, is pre-amplified and fed to a recorder.

3. Amplifier--a Gould Electronics bidirectional bridge amplifier, Model SC 1105, used to pre-amplify the total of the output voltages from the displacement transducers. After amplification, the final voltage output is sent to the strip chart recorder.

Chart Recorder

A two-channel Sargent-Welch strip chart recorder, Model DSRG-2, separately monitored the voltage output of the strain gage load cell and the pre-amplified output of the displacement transducers and plotted the respective output traces on the chart. Calibration factors converted the millivolt readings from the chart to applied load in pounds and to horizontal deformation of the specimen in microinches.

Specimen Positioning Devices

An assembly consisting of a stand or holder and a yoke holds the test specimen and positions it for applying the pulsating load across a vertical diameter (see Figure 29). The specimen rests on the bottom strip, 180 degrees from the top strip which transmits the load to the specimen. The displacement transducers are mounted in a yoke which is clamped to the specimen with four rubber faced clamping screws.

Systems Calibration Procedures

Loading System Calibration

1. Place a proving ring with a known constant beneath the load cell on the testing table.

2. Adjust the regulator to an air pressure of 5.0 psi as indicated on the gage, start the repetitive loading system and apply a pulsating load to the proving ring.

3. Record the proving ring dial gage reading and the millivolt output from the recorder.

4. Increase the air pressure of the loading system in 5.0 psi increments recording the data as in step 3 at each incremental increase.

5. Construct a curve plotting the proving ring load in lbs (as abscissa) versus the load cell output in millivolts (as ordinate) for a given span setting on the recorder.

6. From the curve developed in step 5, determine the load cell output in millivolts corresponding to the desired load in pounds on the specimen. The pulsating loads used in this study were 25, 50 and 75 lbs.

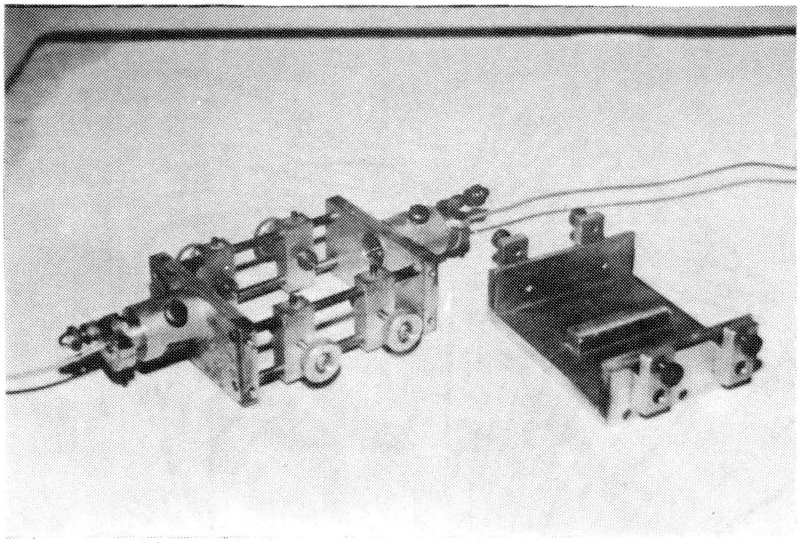


Figure 28. Specimen Holding Device (Yoke and Holder)

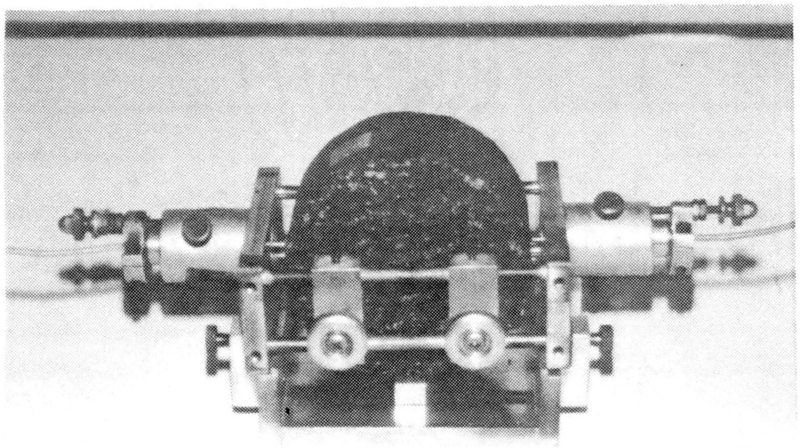


Figure 29. Transducer's Tips Touching Two Opposite Sides of a Foamix Specimen

Transducers Calibration

The purpose of the transducer calibration was to develop a factor relating the transducer tip displacement in microinches to the amplified voltage output as recorded, in millivolts, by the second channel of the chart recorder. The two UC-3 transducers used in this study were calibrated using the following procedure:

1. The input and output leads of the two transducers are hooked in parallel and connected to the amplifier. Connect the amplifier output to the second channel of the chart recorder. The amplifier output then represents the sum of the output from the two transducers, after amplification.

2. Mount one of the transducers in the calibration assembly, as shown in Figure 30, with the transducer's tip barely touching the tip of the micrometer.

3. Turn the micrometer barrel until the recorder pen responds. Set the pen back to zero.

4. Continue turning the micrometer barrel slowly to obtain 1000.0 microinches movement or displacement, as indicated by the micrometer head, and record the change in output on the chart recorder. Repeat for different millivolts span settings of the recorder, if needed.

5. Repeat steps 2 through 4, using the other transducer. Average the millivolt output readings from the transducers. Calculate a factor to convert the millivolt reading from the strip recorder to deformation in microinches by dividing 1000.0 microinches by the average output in millivolts.

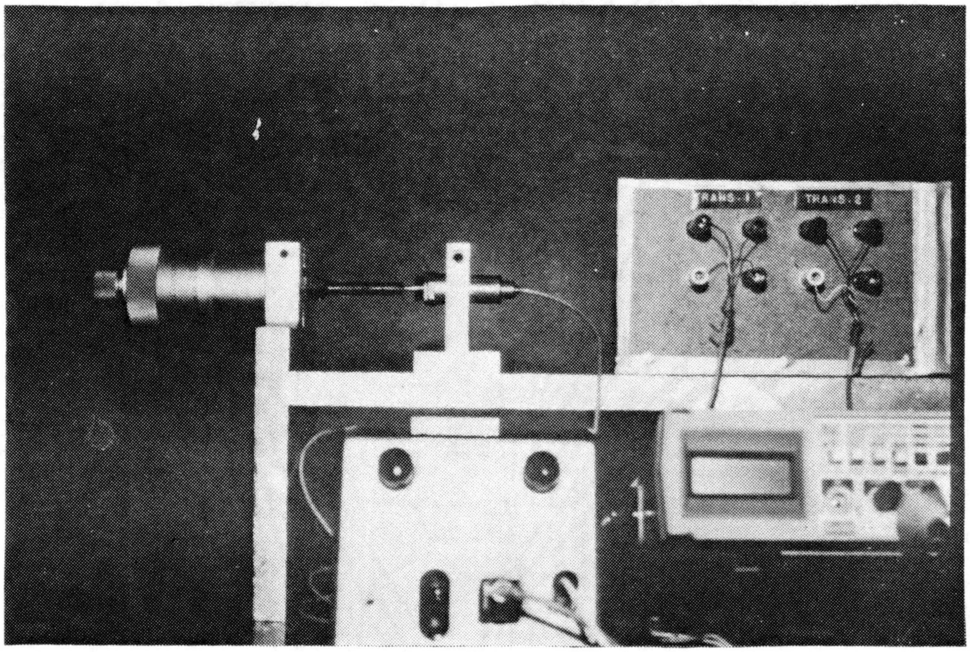


Figure 30. Transducer Calibration Assembly

APPENDIX D

VANE SHEAR TEST-DETAILED DESCRIPTION OF APPARATUS AND SHEAR STRESS CALCULATIONS

Vane Shear Test Apparatus

The vane shear test apparatus, similar to that used by Marias (43) and Acott (8), was manufactured by the Oklahoma State University College of Engineering's Research and Development Laboratory. The apparatus consisted of the following parts:

Tripod Stand

A 24.5 in. (622.0 mm) tall metal tripod stand with 4.0 in. (102.0 mm) diameter base plates welded to the bottom of each leg. Holes 3/16 in. (4.75 mm) in diameter in these base plates facilitate securing the stand in position for testing. A 6.0 in. (152.0 mm) long metal guide sleeve, with its long axis positioned vertical to the plane on which the tripod rests, is centered in the head of the stand. Dimensional details of the stand are shown in Figure 31.

Torque Shaft

A 22.0 in. (559.0 mm) long shaft having a nominal diameter of 5/8 in. (16.0 mm) and with a 1/2 in. (13.0 mm) socket on one end to fit the vane cutter shaft. A 1/2 in. (13.0 mm) hexagonal stud on the other end of the shaft accepts the socket of the torque wrench, see Figure 32A. This shaft slides through the guide sleeve in the tripod stand and is positioned vertically during driving and testing.

Anvil

A 6.0 in. (153.0 mm) long metal cylinder which fits over the head of the torque shaft (see Figure 32B). The anvil protects the shaft head while hammering the vane cutter into a sample to be tested.

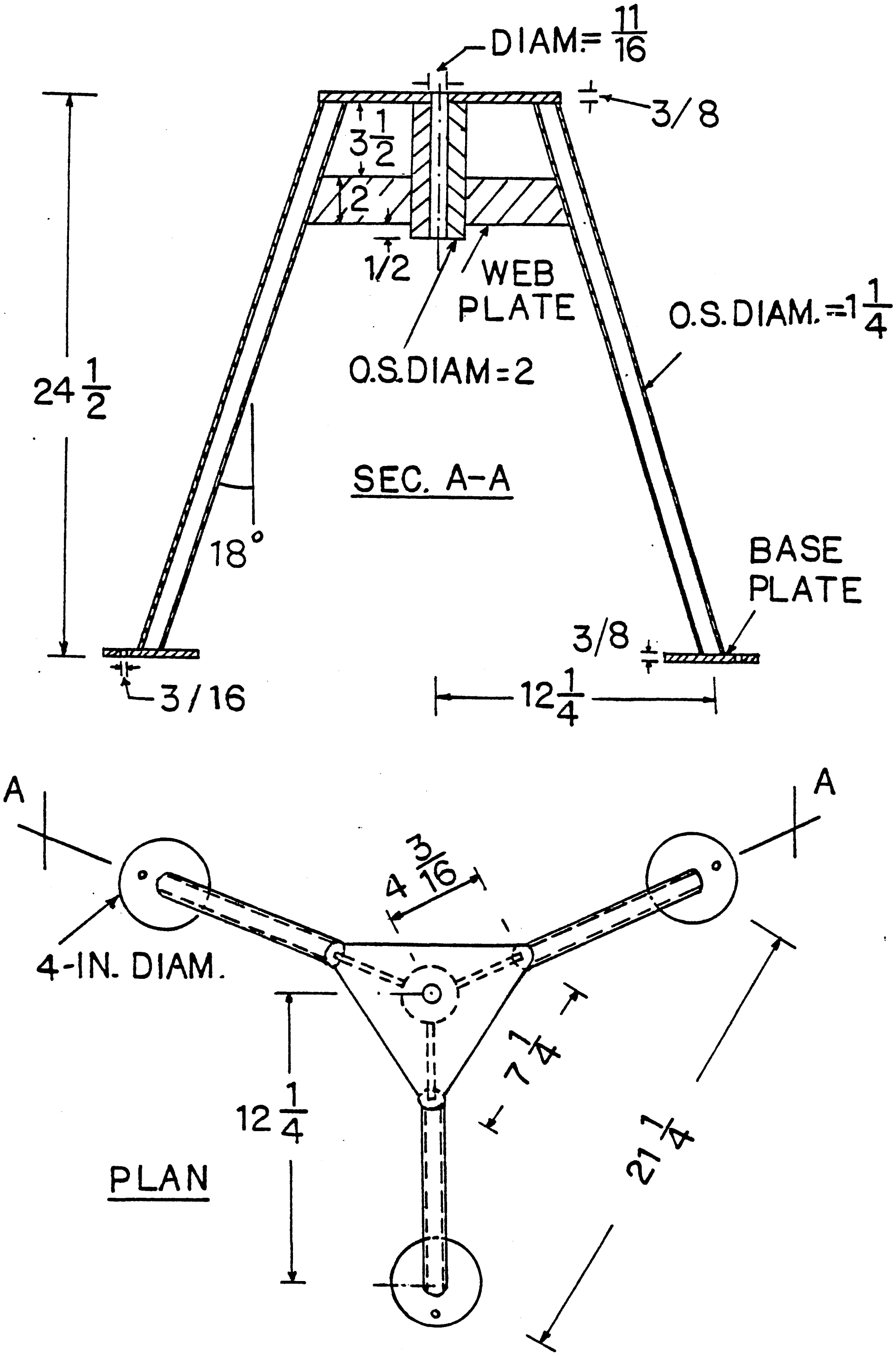
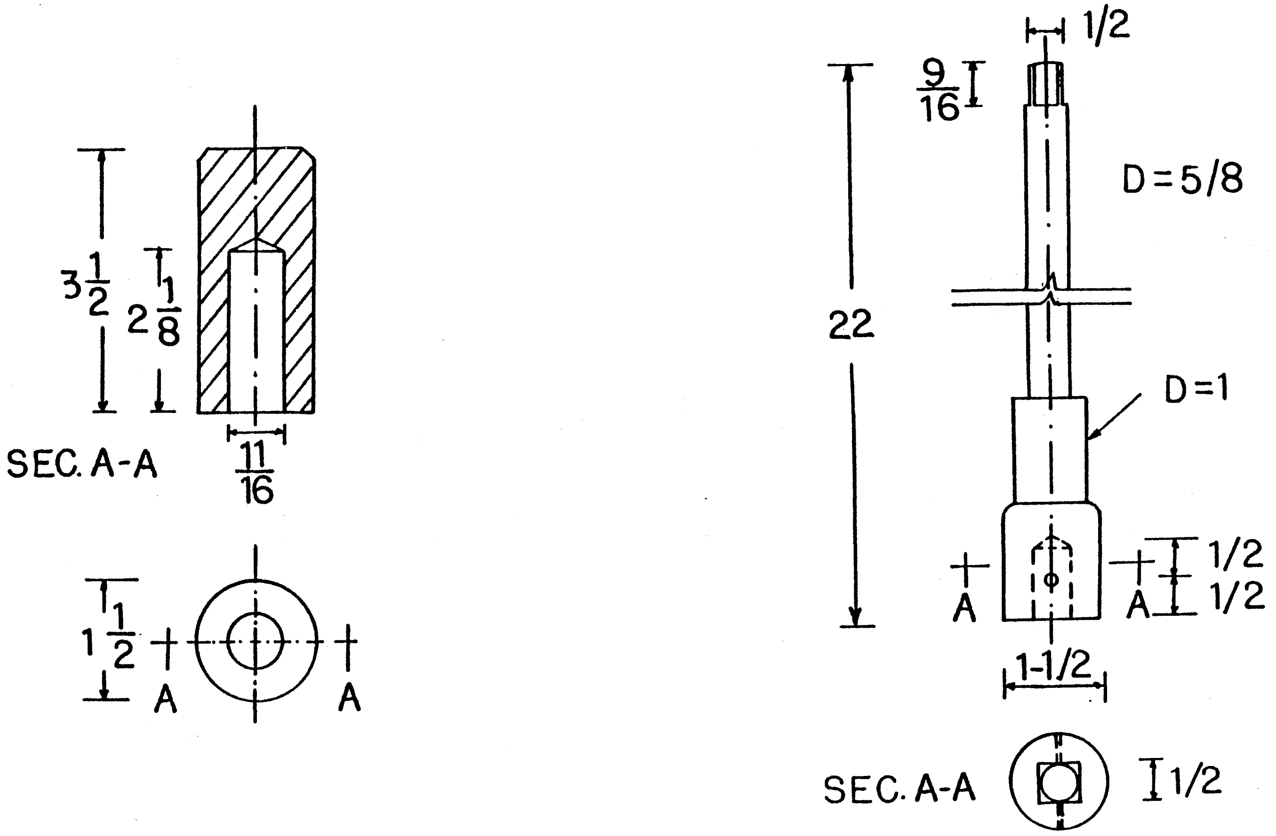


Figure 31. Details of Tripod Stand (Dimensions Shown in Inches)



(B) ANVIL

(A) TORQUE SHAFT

Figure 32. Details of Anvil and Torque Shaft (Dimensions Shown in Inches)

Vane Cutters

The four blade cutting head was machined from a single piece of steel with the configuration shown in Figure 33. The vane cutter is driven into a sample to be tested until the upper edge of the blades is at least 1/2 in. (13 mm) below the surface of the material. Torque is then applied to shear out a cylinder of the material having a diameter and height equal to these of the vane cutter. Three sizes of cutter are used to cover shear strengths ranging from 1.0 to 150.0 psi. Dimensions, constants, and ranges for the three cutter sizes are given in the table of Figure 33.

Torque Wrench

A dial type torque wrench, manufactured by SNAP-ON-TOOLS Corp., with a dial gage calibrated to read directly torque values ranging from 0.0 to 175.0 ft-lb.

Shear Strength Calculations

The shear strength of the tested specimen is calculated using the following relation:

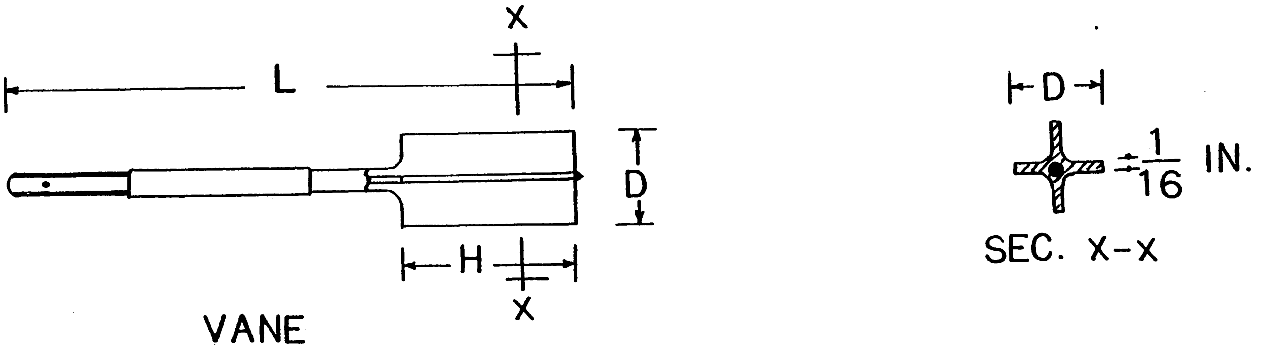
$$S = \frac{T}{K}$$

where:

S = Shear strength, psi

T = Measured torque, ft.-lb.

K = A constant based on the dimensions of the vane, in.^2-ft., calculated as follows:



Vane No.	Torque Range Ft-lb	Shear Strength Range (PSI.)	Vane Dimensions			Vane Constant (K), in^3
			L (in.)	H (in.)	D (in.)	
1	20.0 - 75.0	40.0 - 150.0	9.27	2.40	1.18	0.509
2	10.0 - 52.0	10.0 - 50.0	9.83	2.96	1.50	1.019
3	0.0 - 30.0	0.0 - 15.0	10.66	3.82	1.87	2.034

Figure 33. Vane Cutter and Dimensions of Three Available Vanes

$$K = \frac{\pi}{12} \times \frac{D^2H}{2} \times (1 + \frac{D}{3H})$$

where:

D = Diameter of vane, in, and

H = Height of vane, in.

APPENDIX E

GRAPHIC EXPERIMENTAL DATA

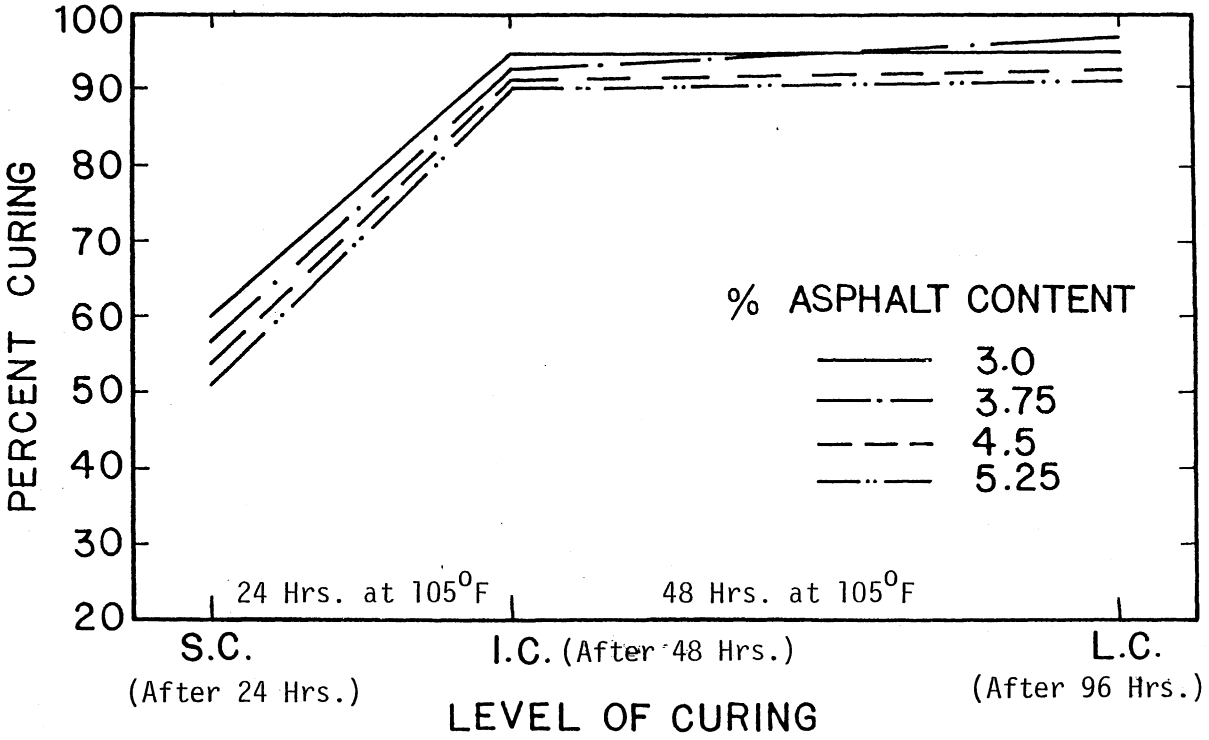


Figure 34. Effect of Curing Level on the Percent of Moisture Lost as Measured by Percent Curing for Foamix (N-1)

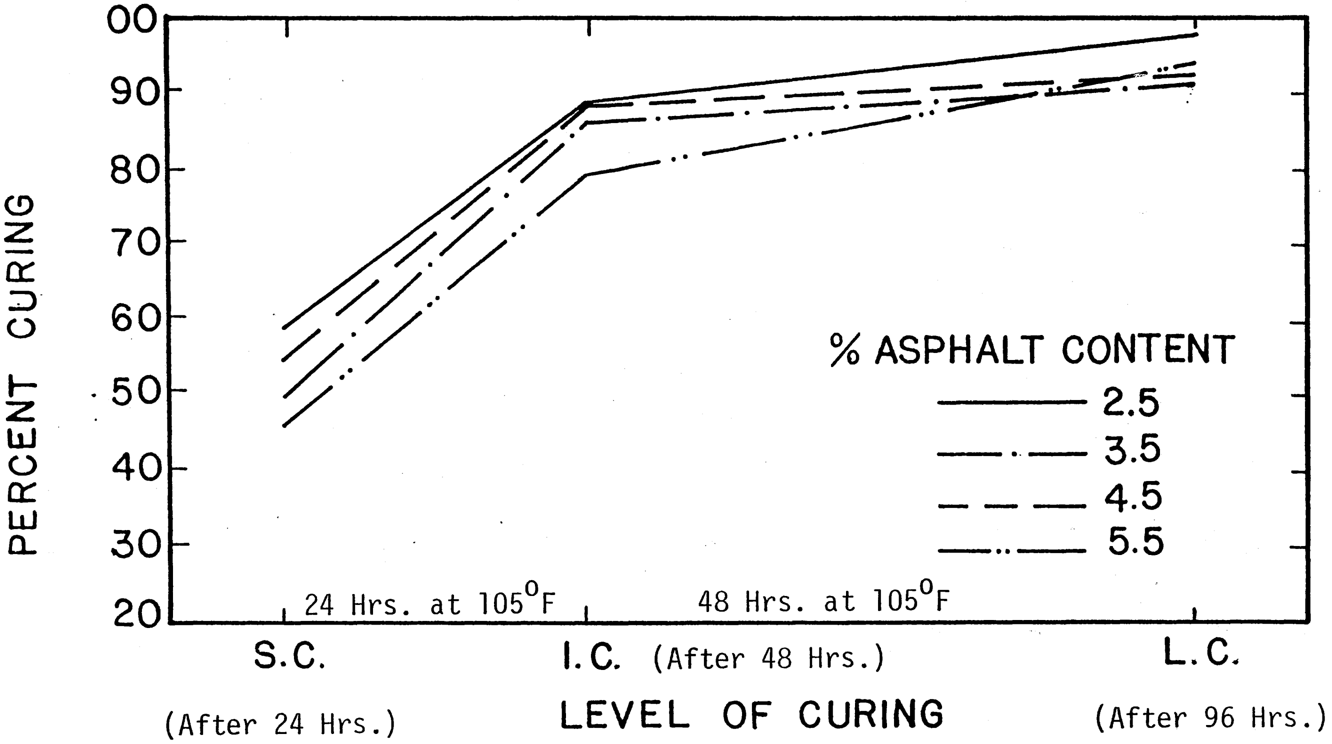


Figure 35. Effect of Curing Level on the Percent Moisture Loss as Measured by Percent Curing for Foamix (B-1)

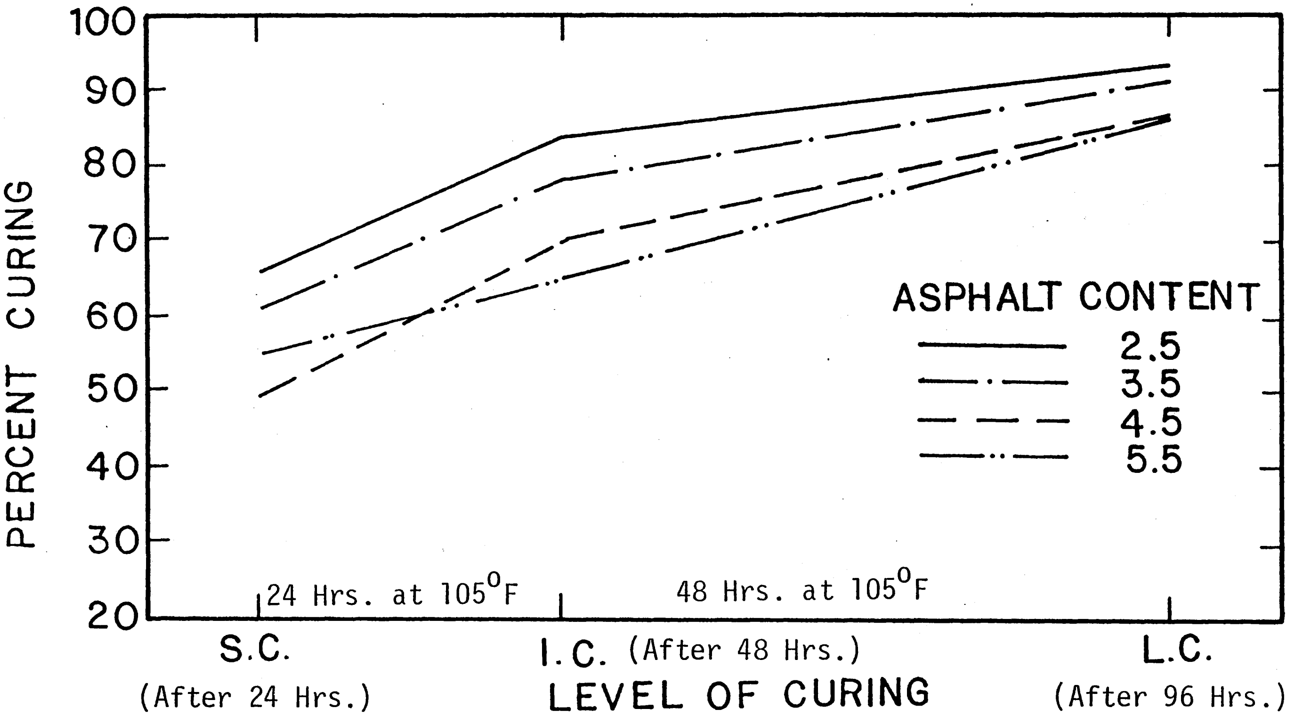


Figure 36. Effect of Level of Curing on the Percent of Moisture Lost as Measured by Percent Curing for Foamix (C-1)

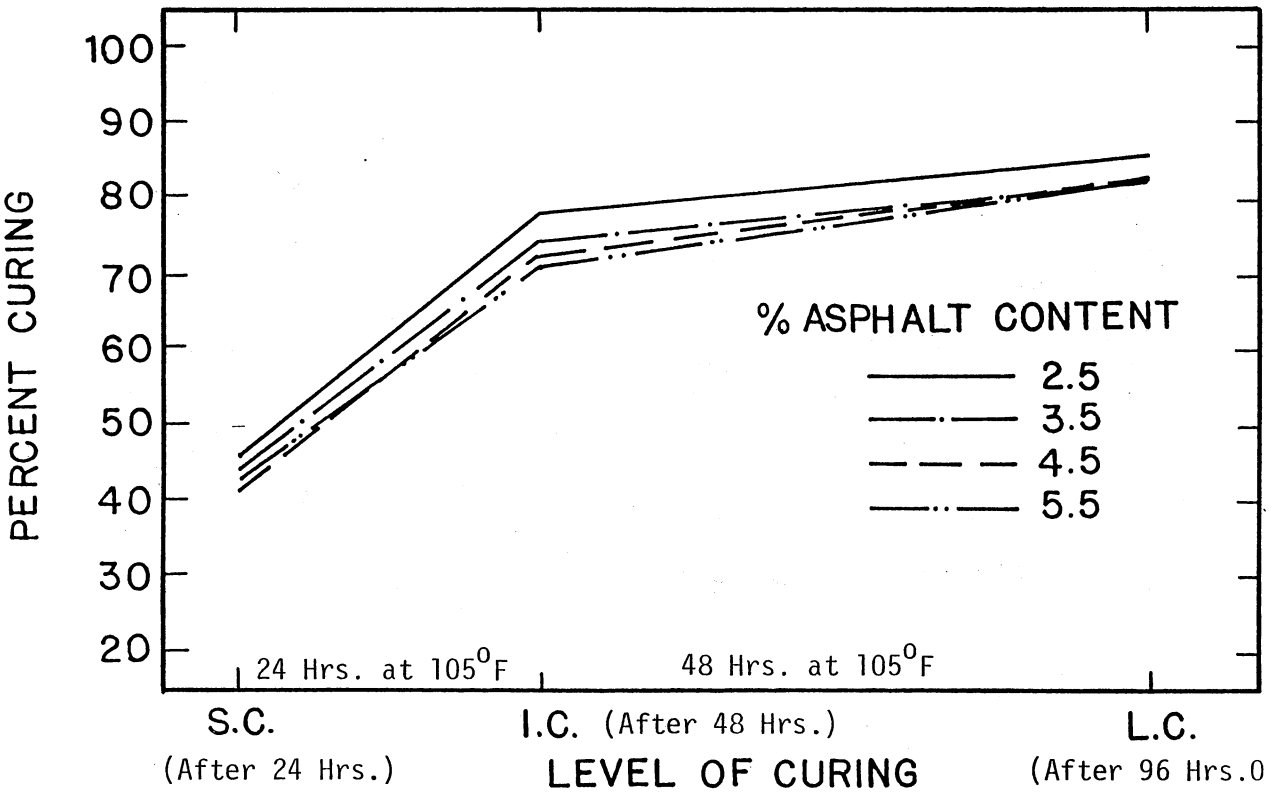


Figure 37. Effect of Curing Level on the Percent Moisture Lost as Measured by Percent Curing for Foamix (C-2)

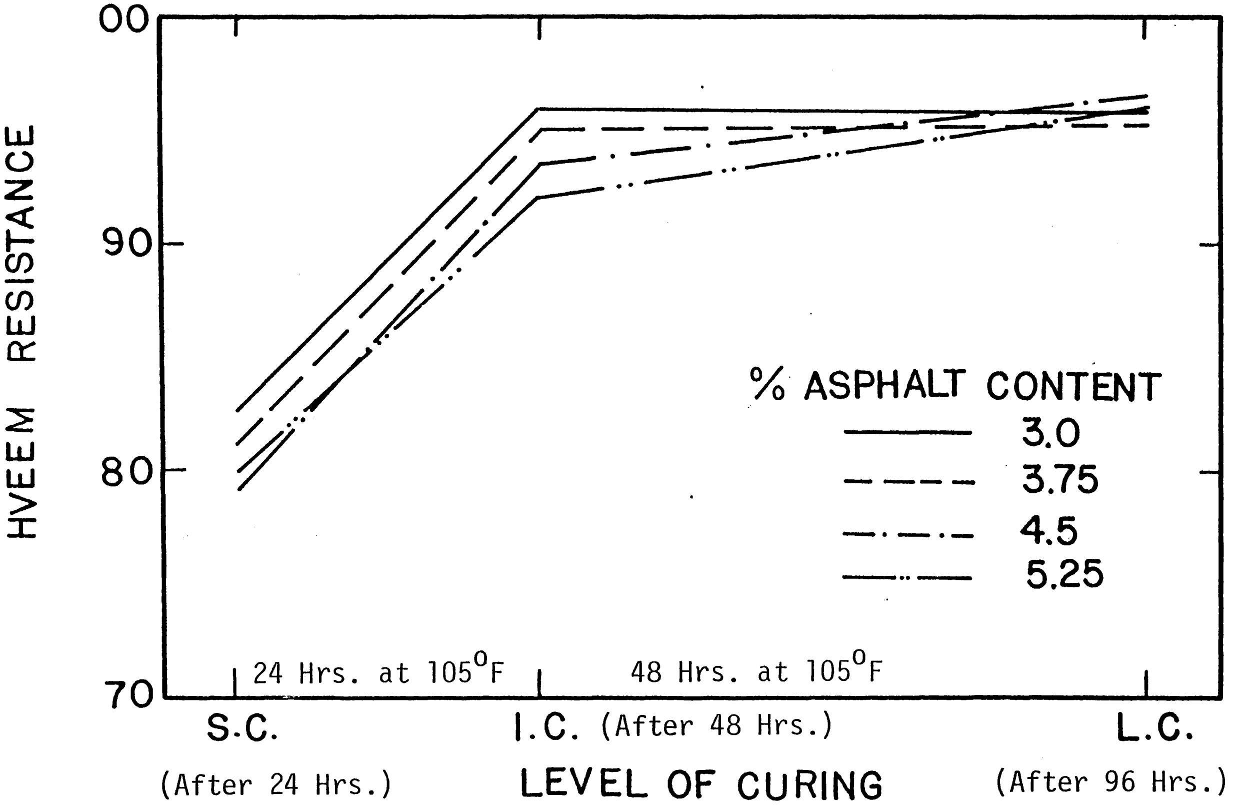


Figure 38. Effect of Curing Level on Hveem Resistance of Foamix (N-1)

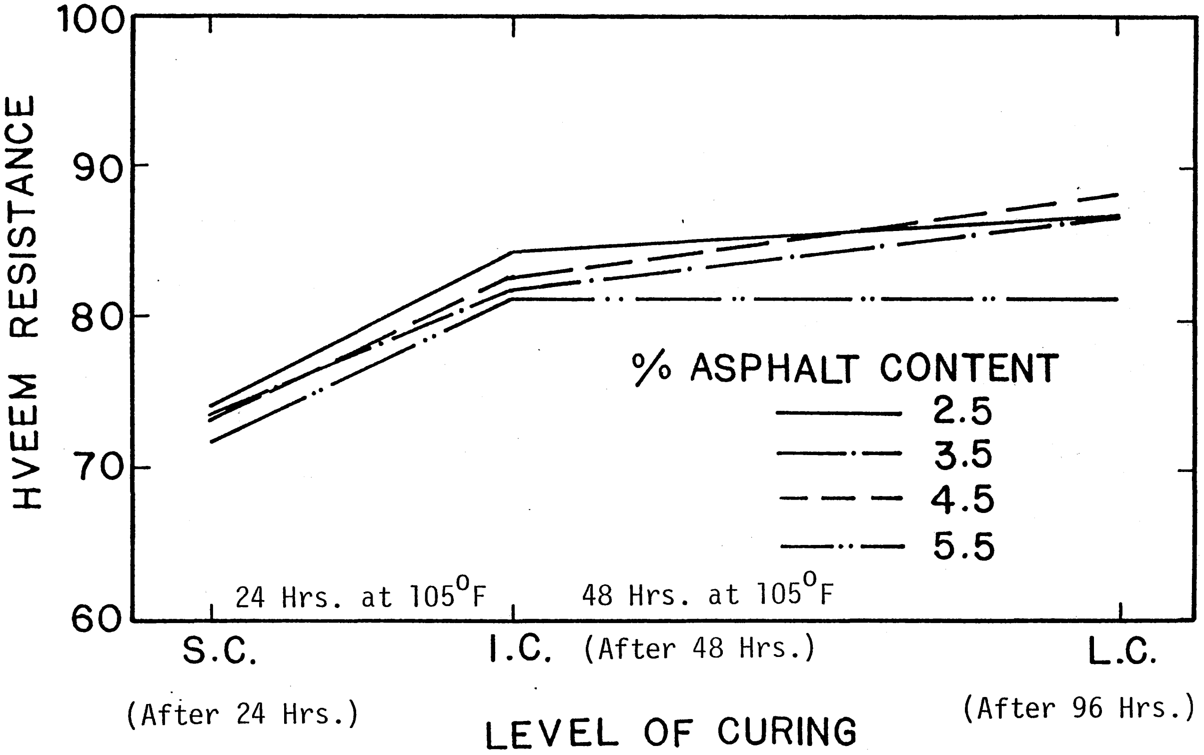


Figure 39. Effect of Curing Level on Hveem Resistance of Foamix (B-1)

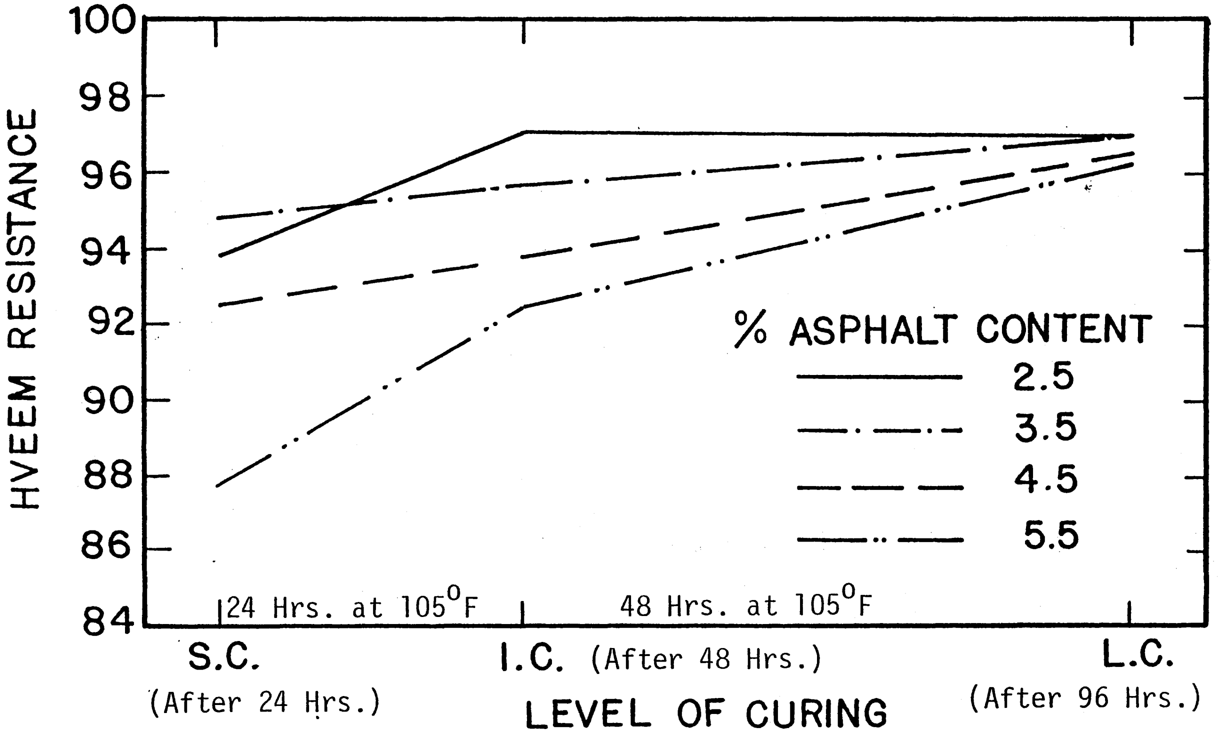


Figure 40. Effect of Curing Level on Hveem Resistance of Foamix (C-1)

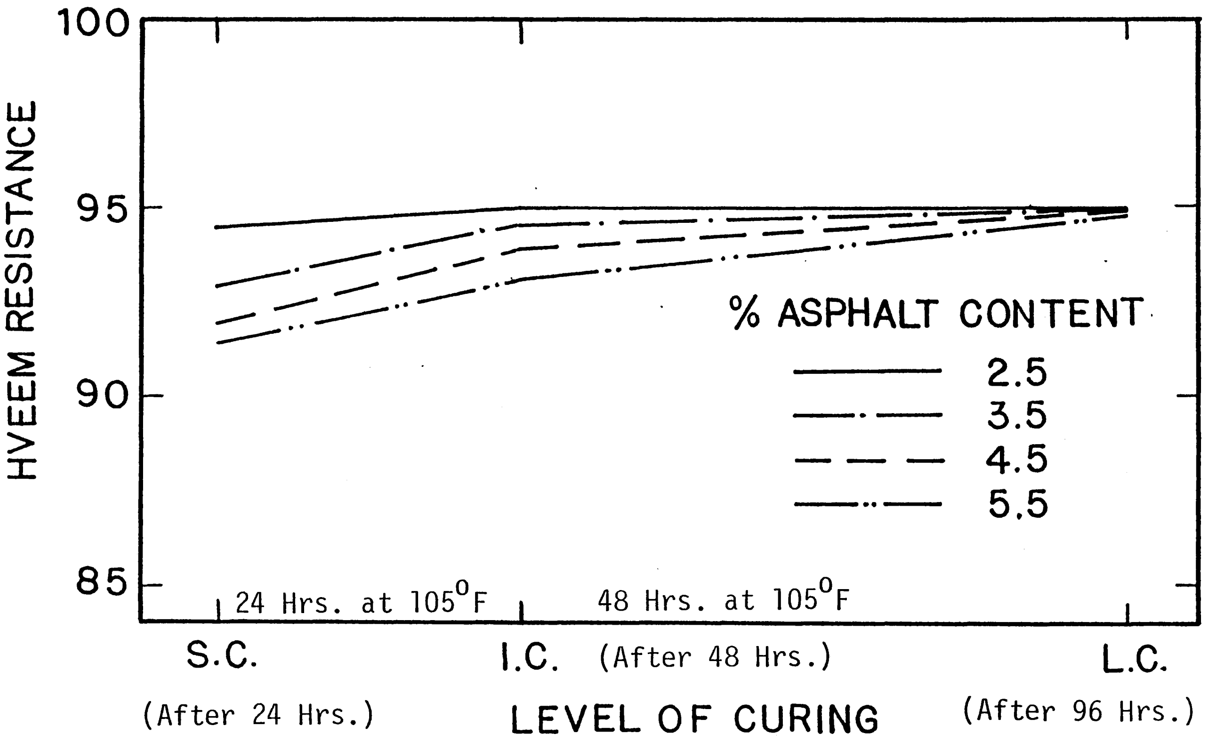


Figure 41. Effect of Curing Level on Hveem Resistance of Foamix (C-2)

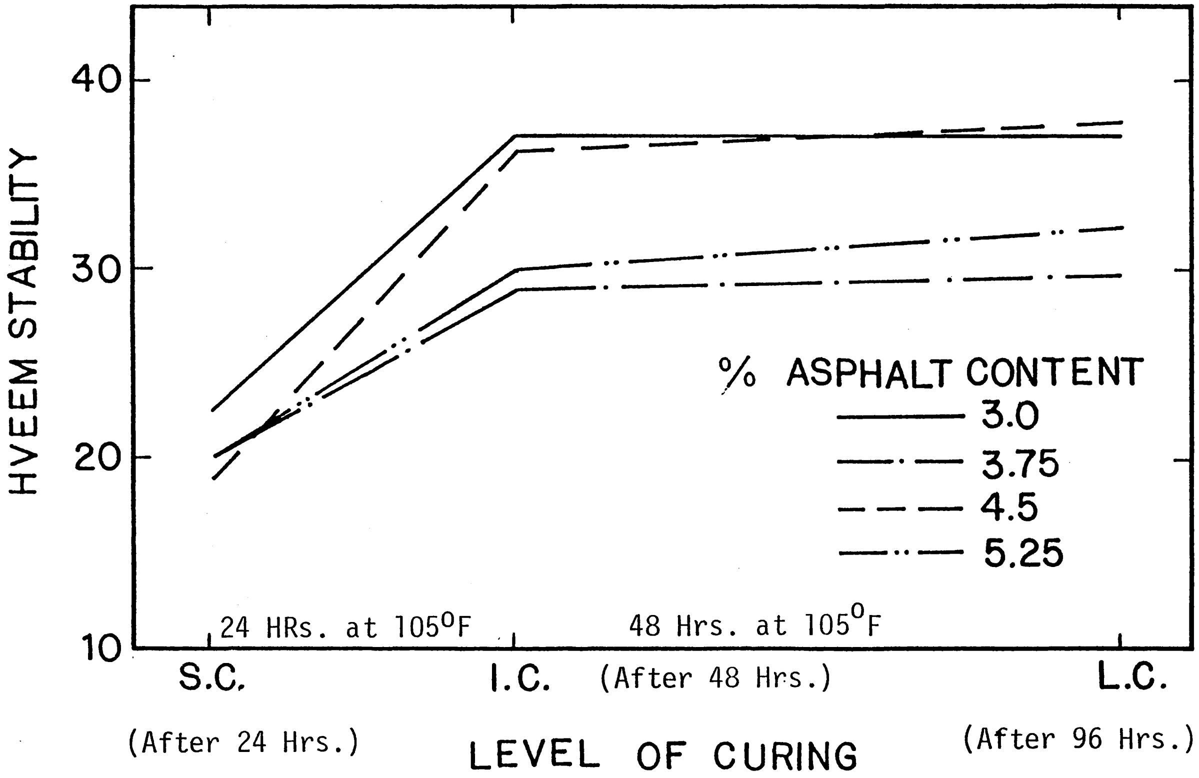


Figure 42. Effect of Curing Level on Hveem Stability of Foamix (N-1)

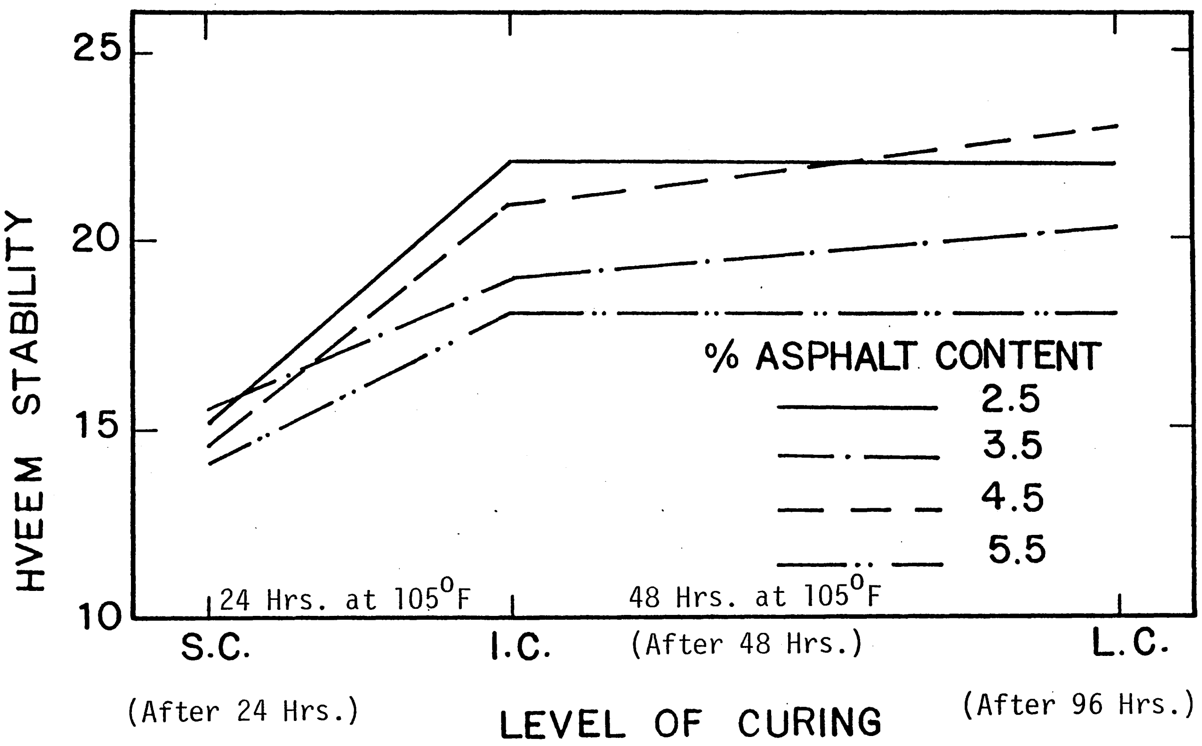


Figure 43. Effect of Curing Level on Hveem Stability of Foamix (B-1)

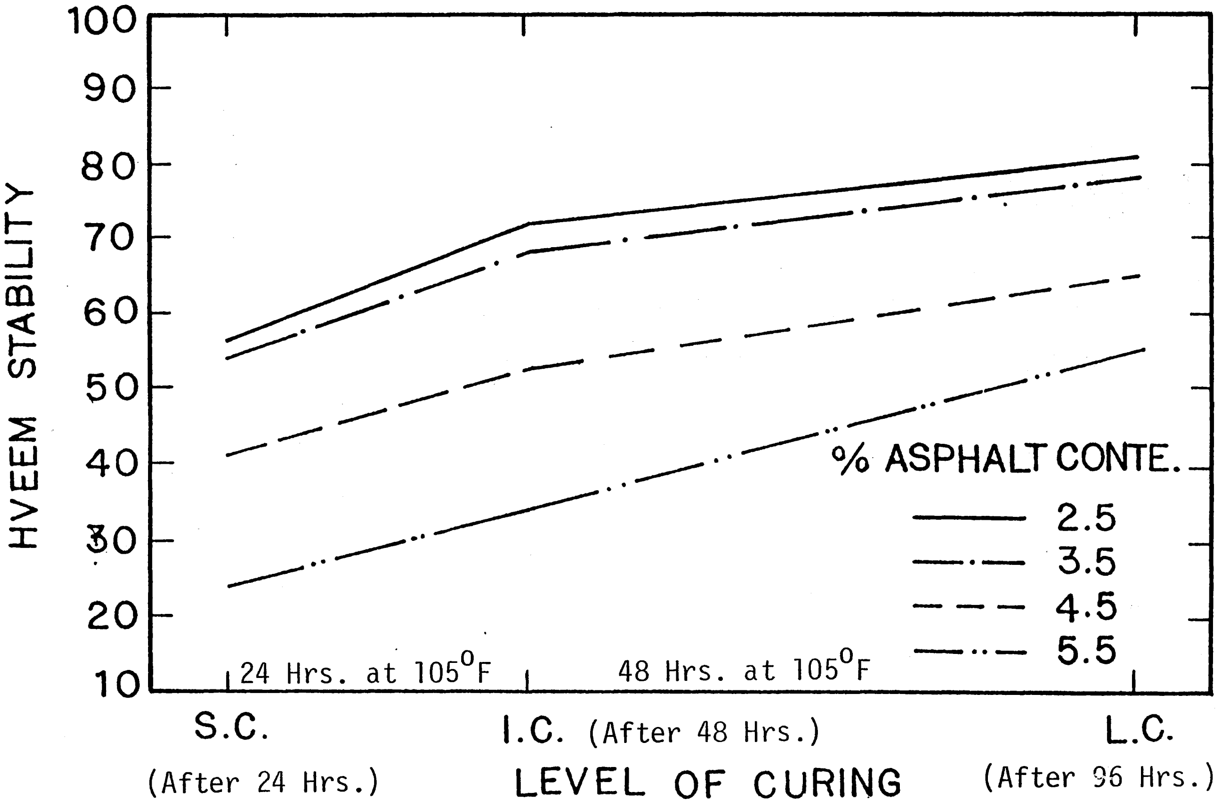


Figure 44. Effect of Curing Level on Hveem Stability of Foamix (C-1)

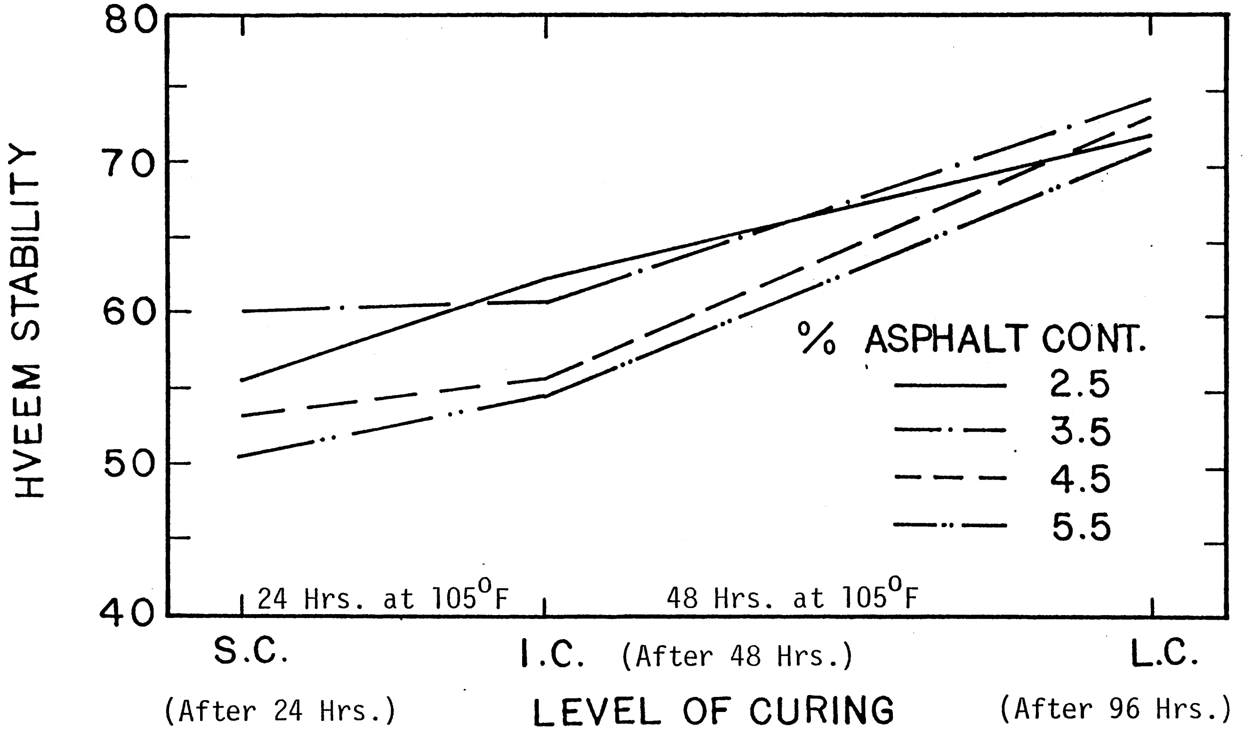


Figure 45. Effect of Curing Level on Hveem Stability of Foamix (C-2)

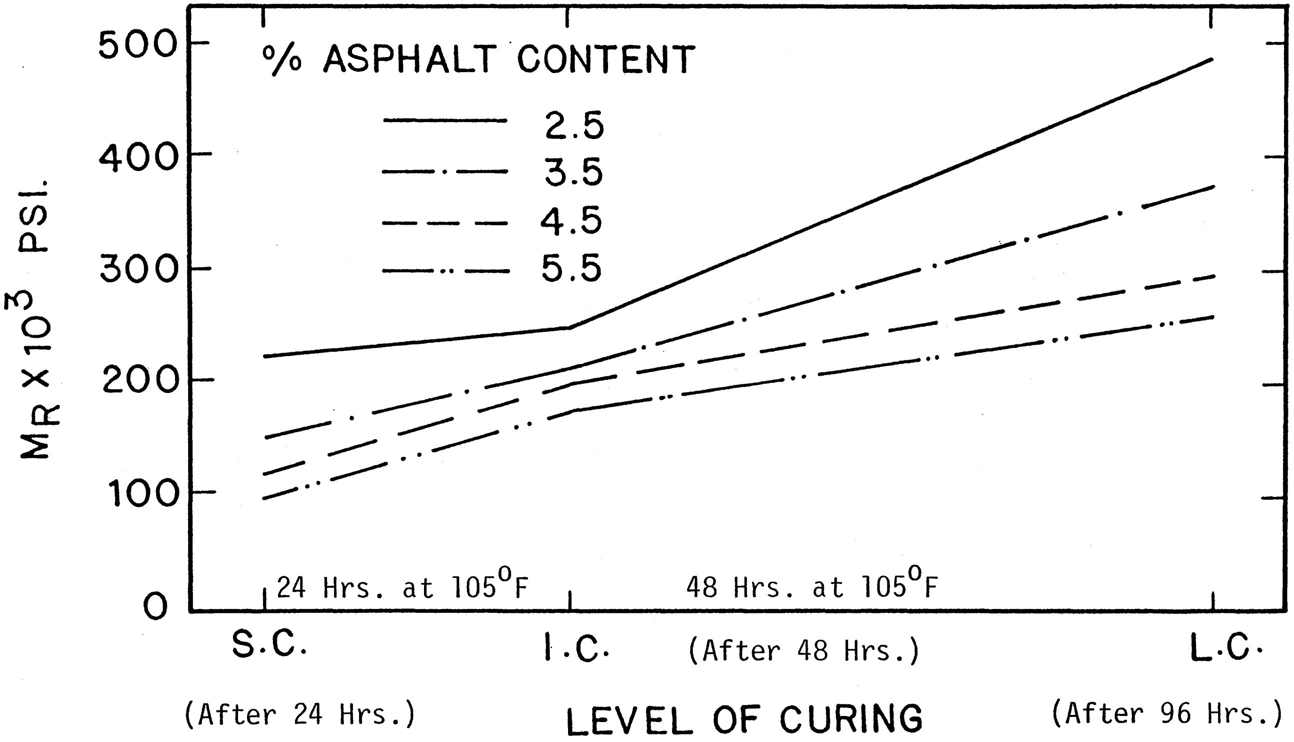


Figure 46. Effect of Curing Level on the Resilient Modulus of Foamix (C-1)

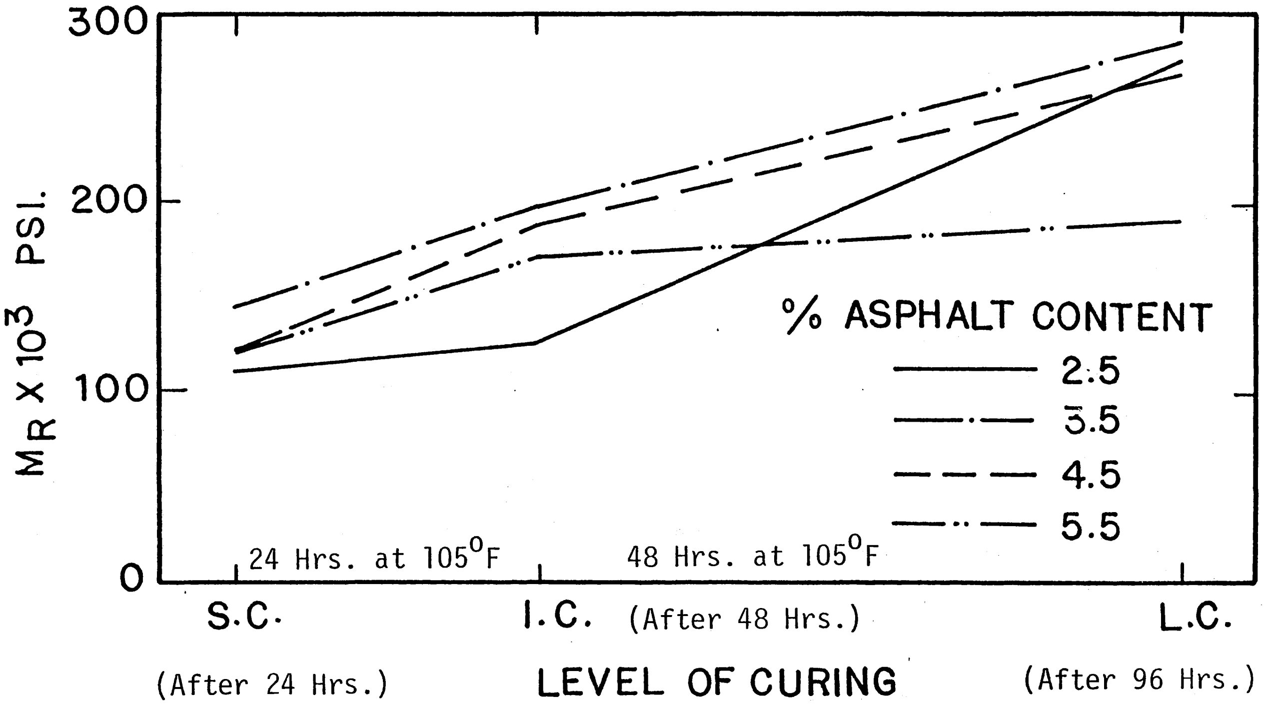


Figure 47. Effect of Curing Level on the Resilient Modulus of Foamix (C-2)

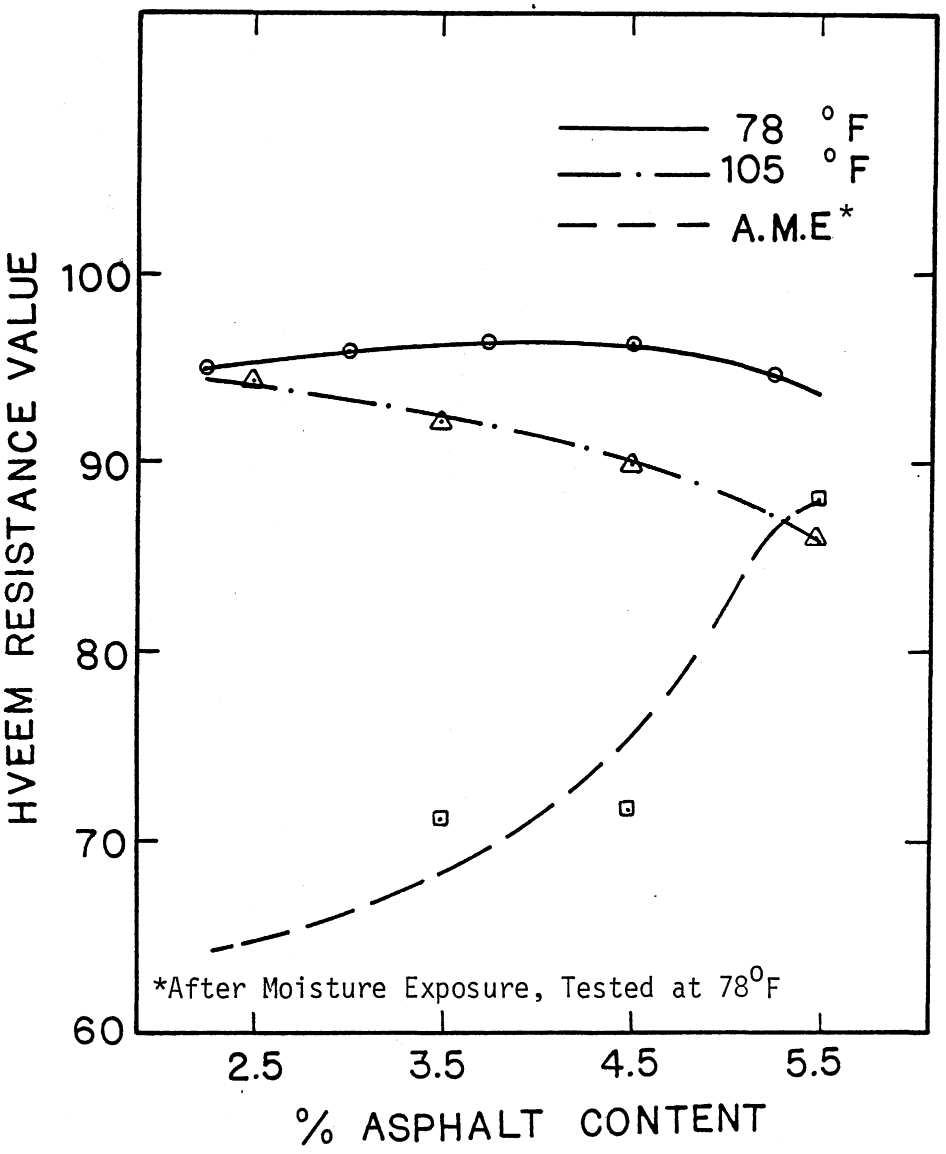


Figure 48. Hveem Resistance Versus Asphalt Content for Foamix (N-1) After Long Curing

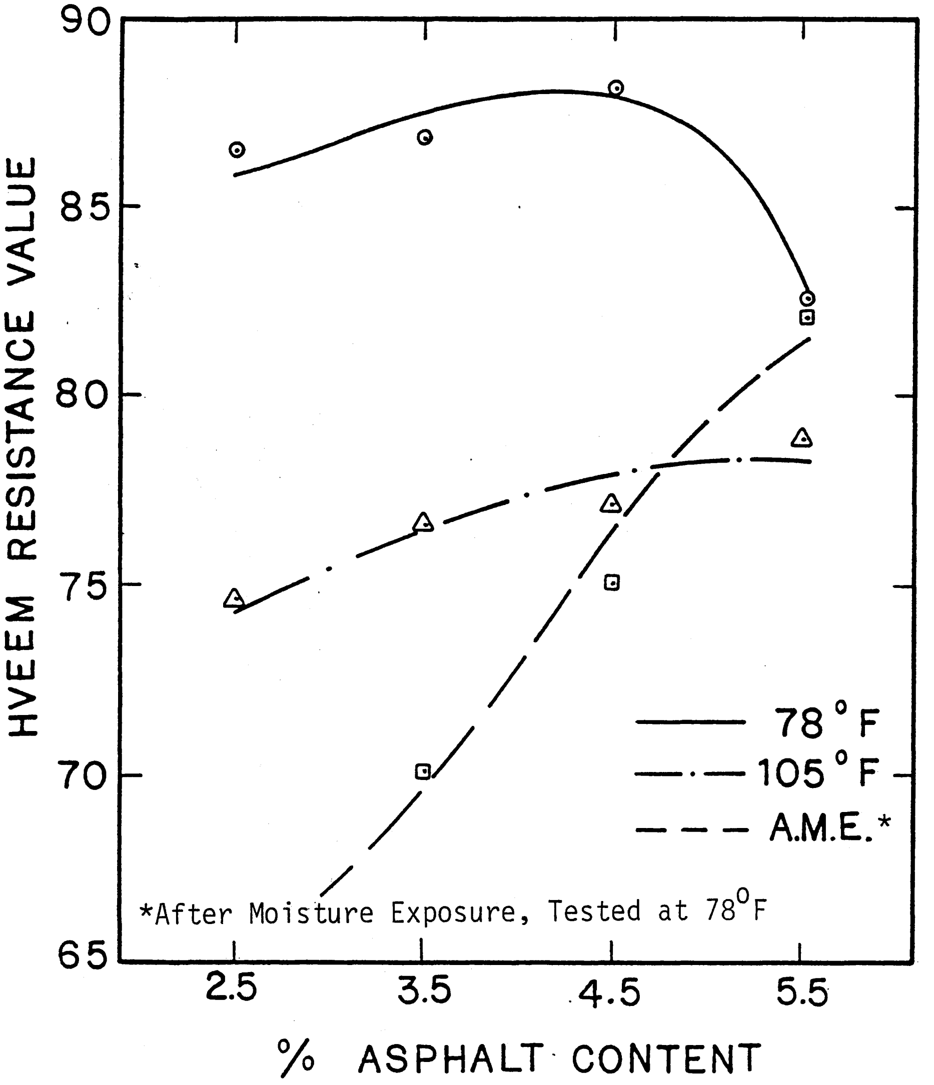


Figure 49. Hveem Resistance Versus Asphalt Content for Foamix (B-1) After Long Curing

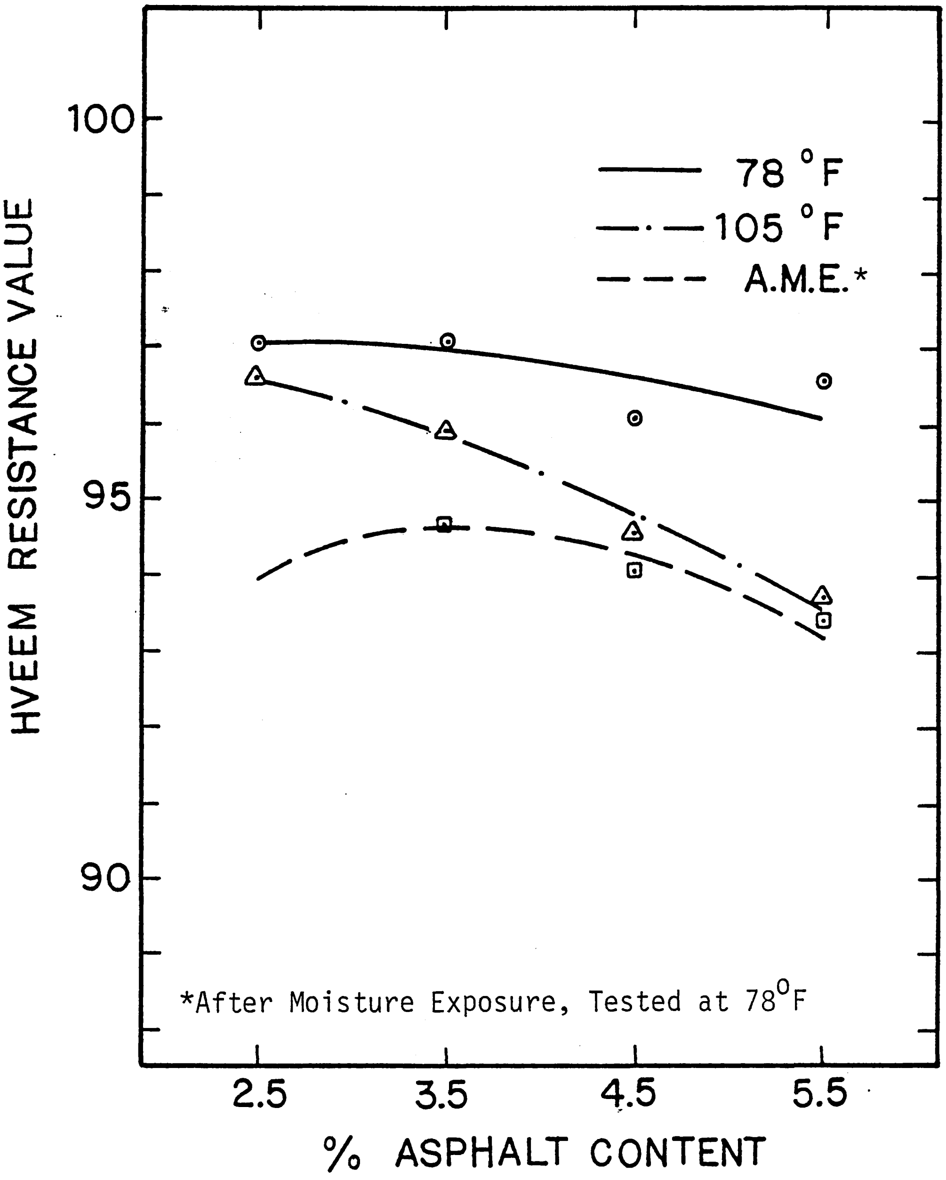


Figure 50. Hveem Resistance Versus Asphalt Content for Foamix (C-1) After Long Curing

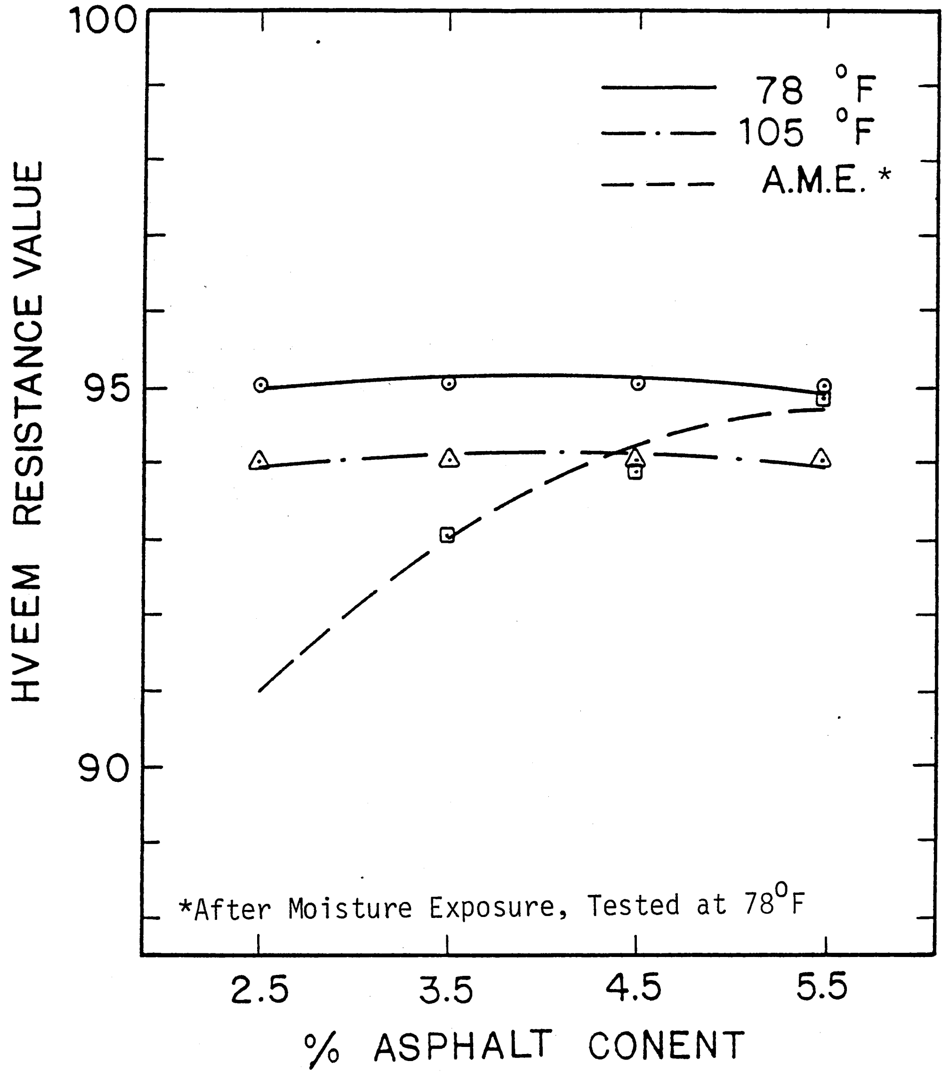


Figure 51. Hveem Resistance Versus Asphalt Content for Foamix (C-2) After Long Curing

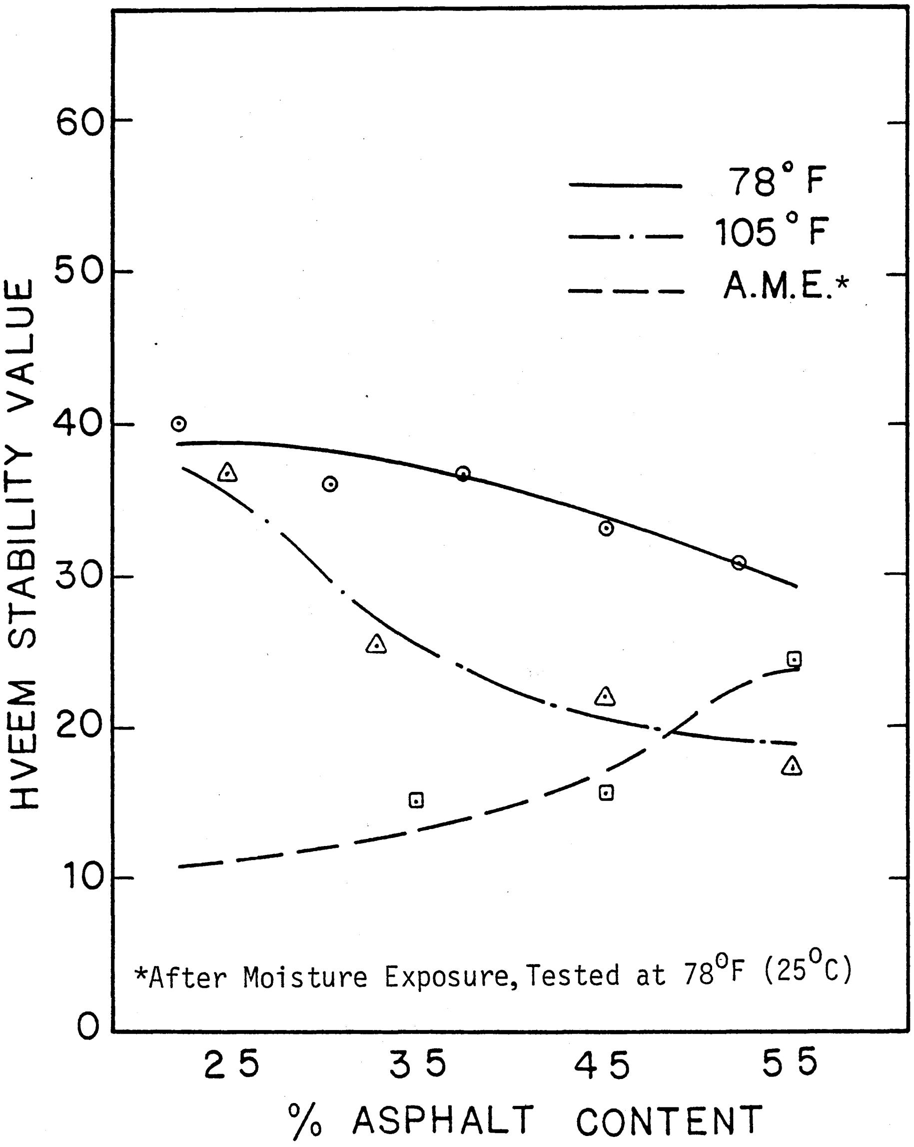


Figure 52. Hveem Stability Versus Asphalt Content for Foamix (N-1) After Long Curing

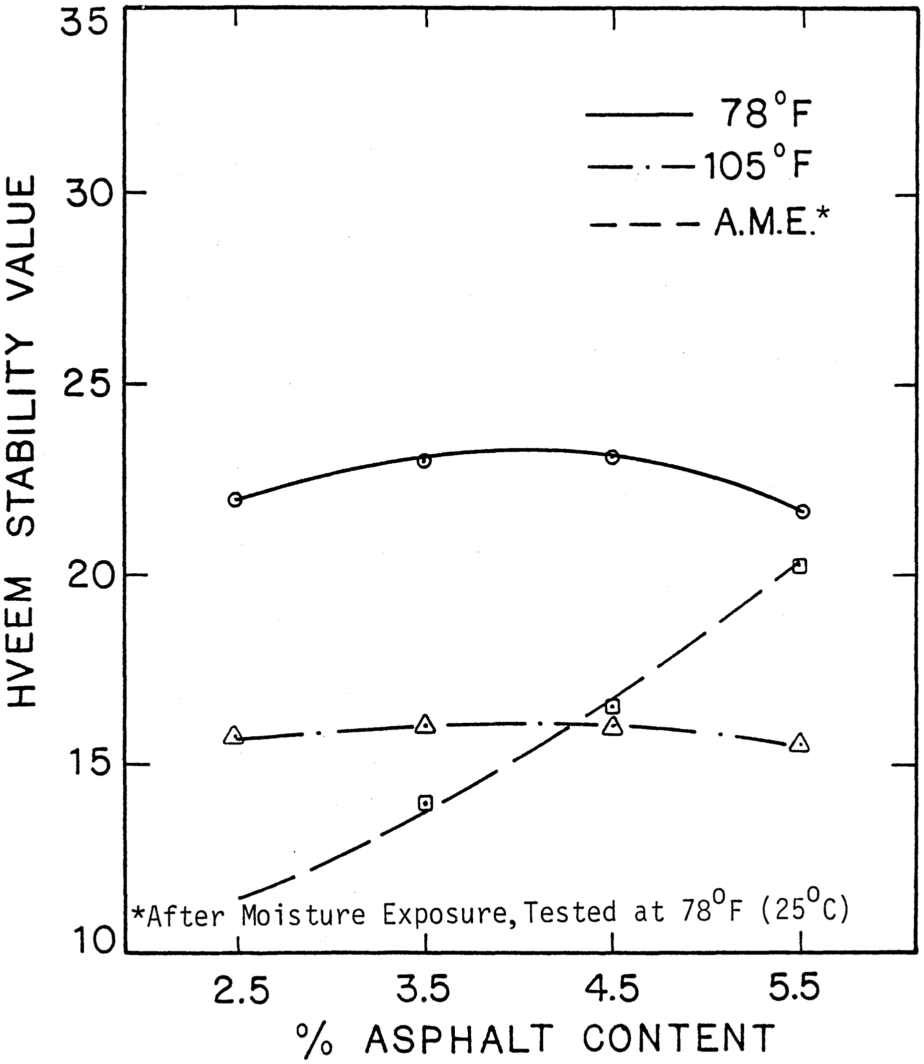


Figure 53. Hveem Stability Versus Asphalt Content for Foamix (B-1) After Long Curing

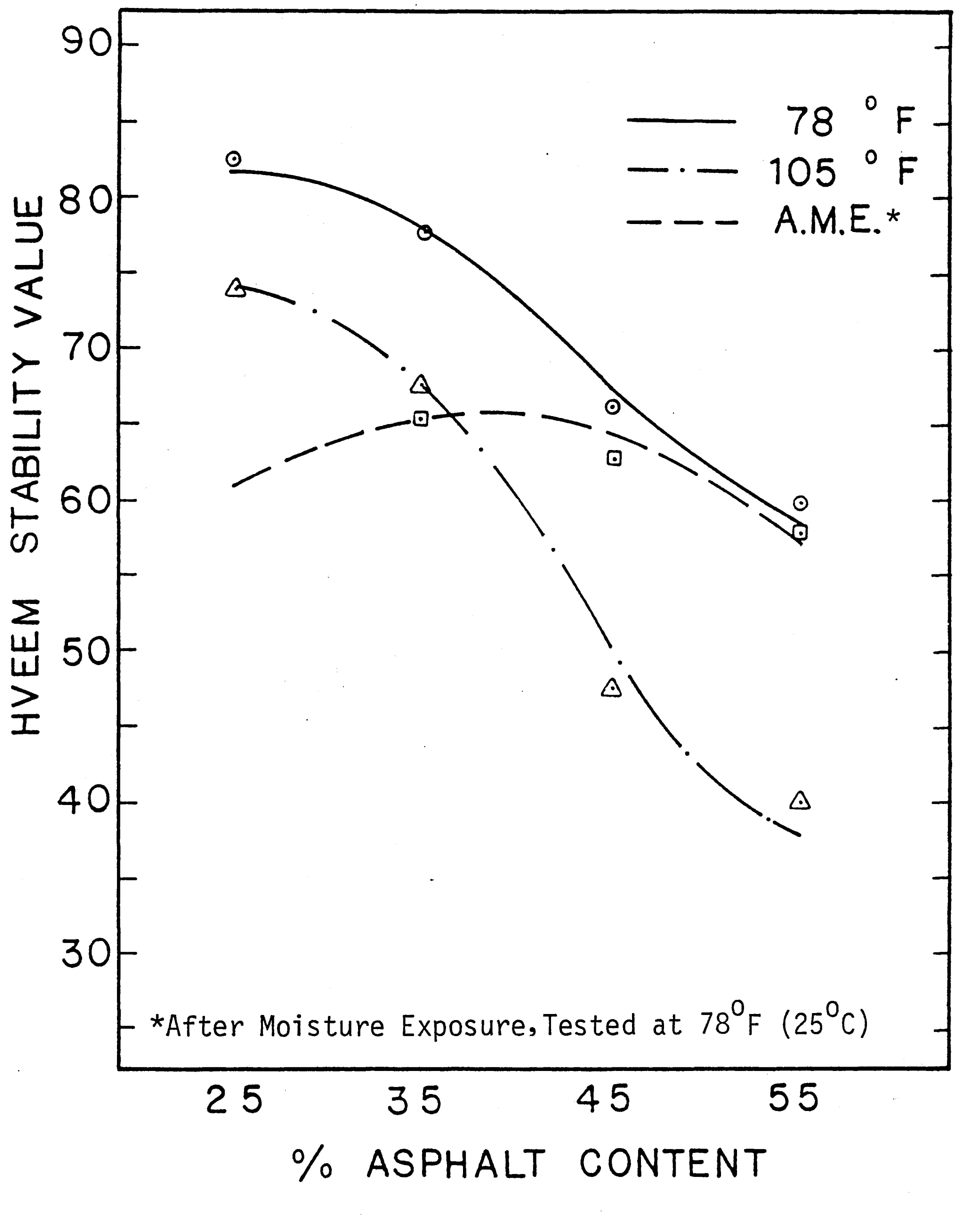


Figure 54. Hveem Stability Versus Asphalt Content for Foamix (C-1) After Long Curing

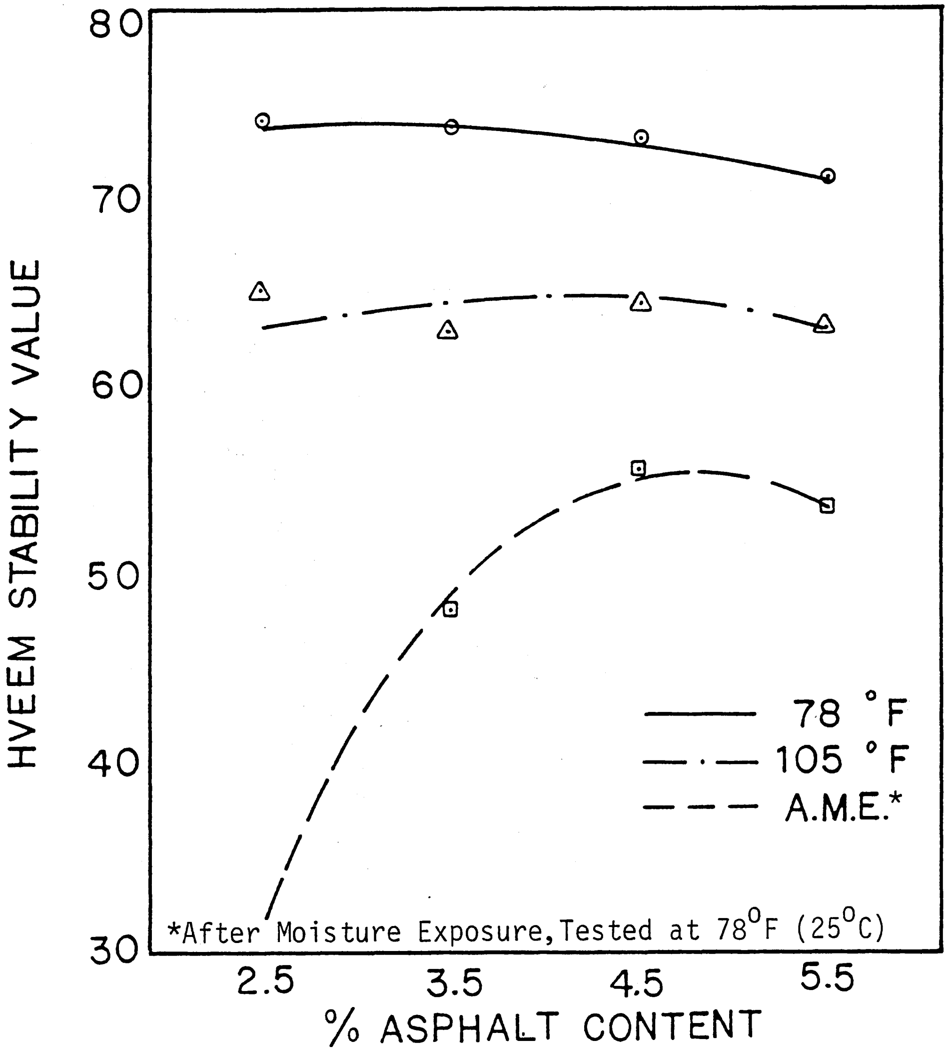


Figure 55. Hveem Stability Versus Asphalt Content for Foamix (C-2) After Long Curing

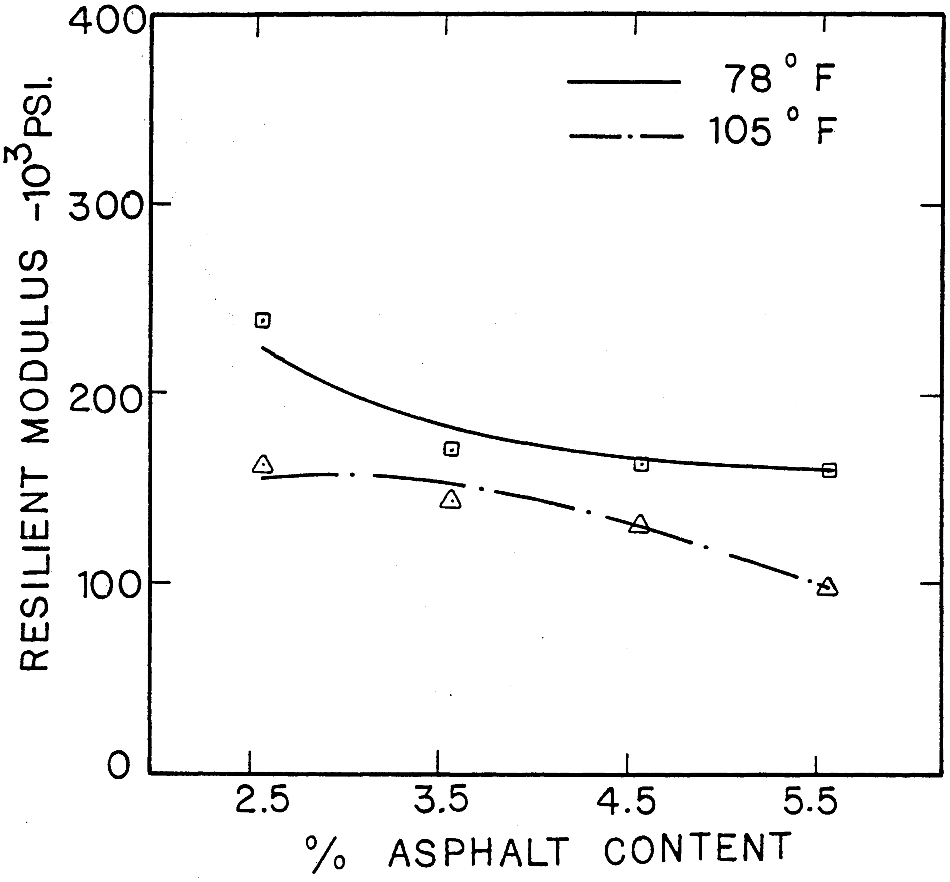


Figure 56. Relation Between Resilient Modulus and Asphalt Content for Foamix (N-1)

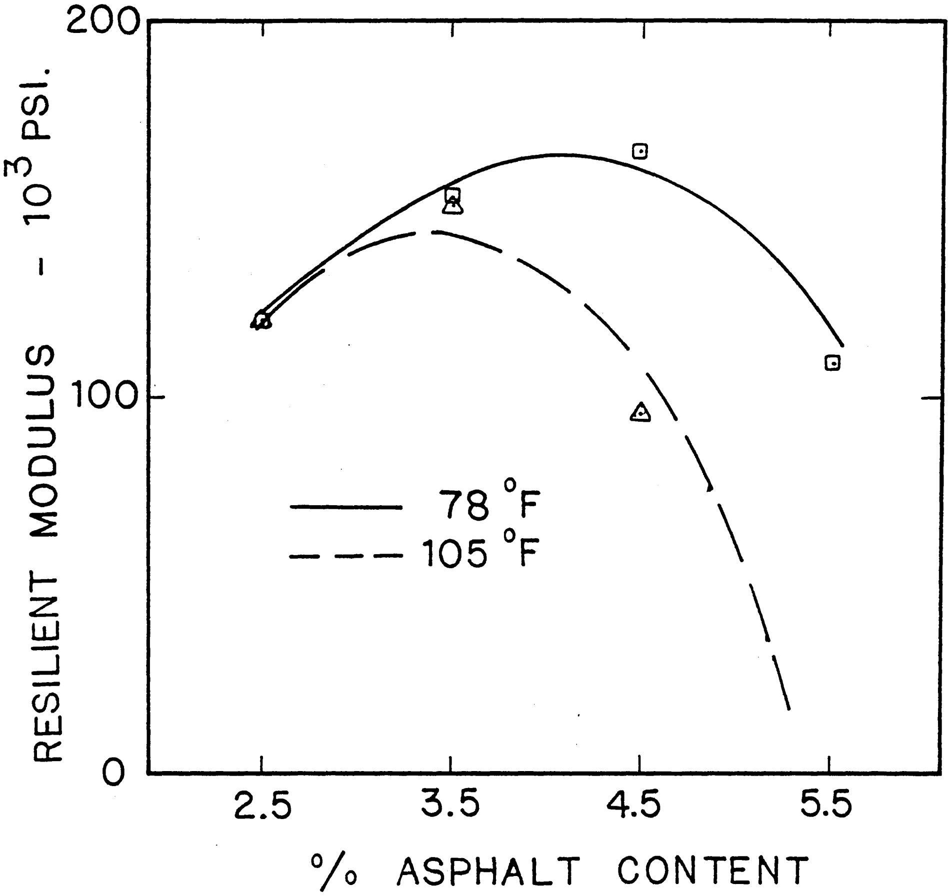


Figure 57. Resilient Modulus Versus Asphalt Content for Foamix (B-1)

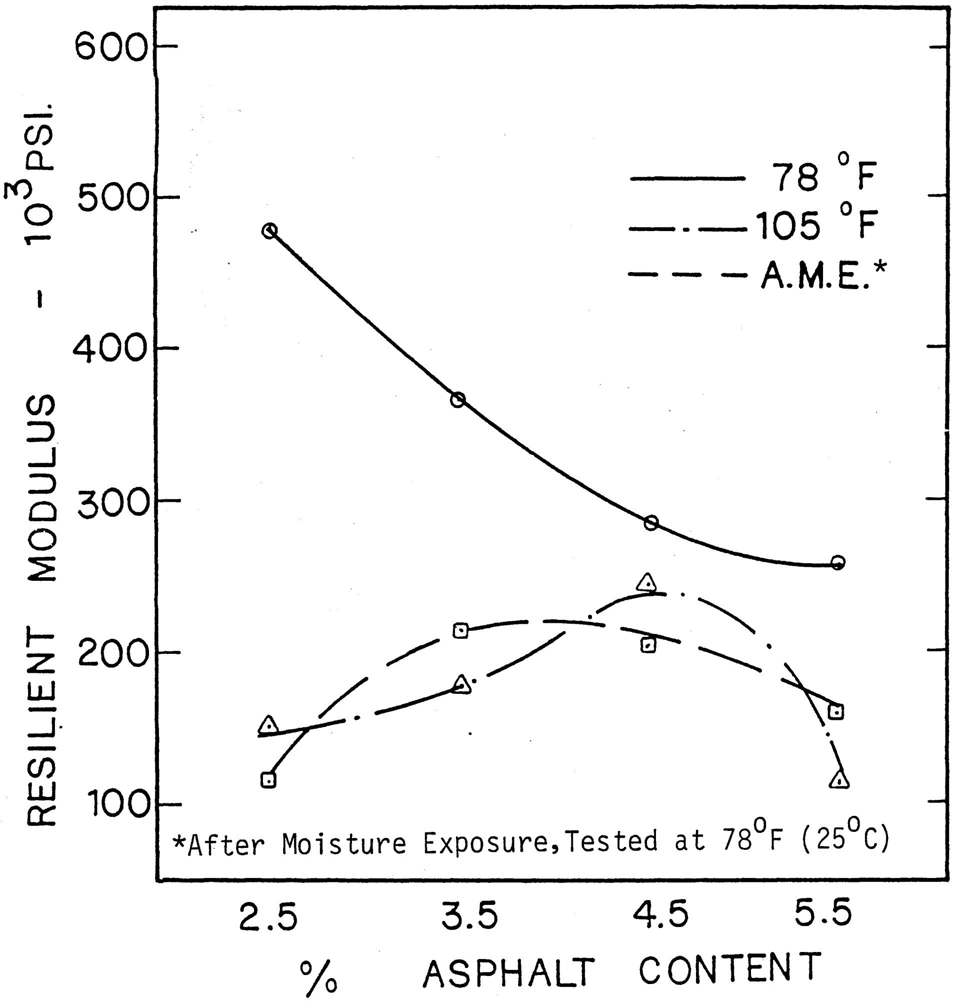


Figure 58. Resilient Modulus Versus Asphalt Content for Foamix (C-1)

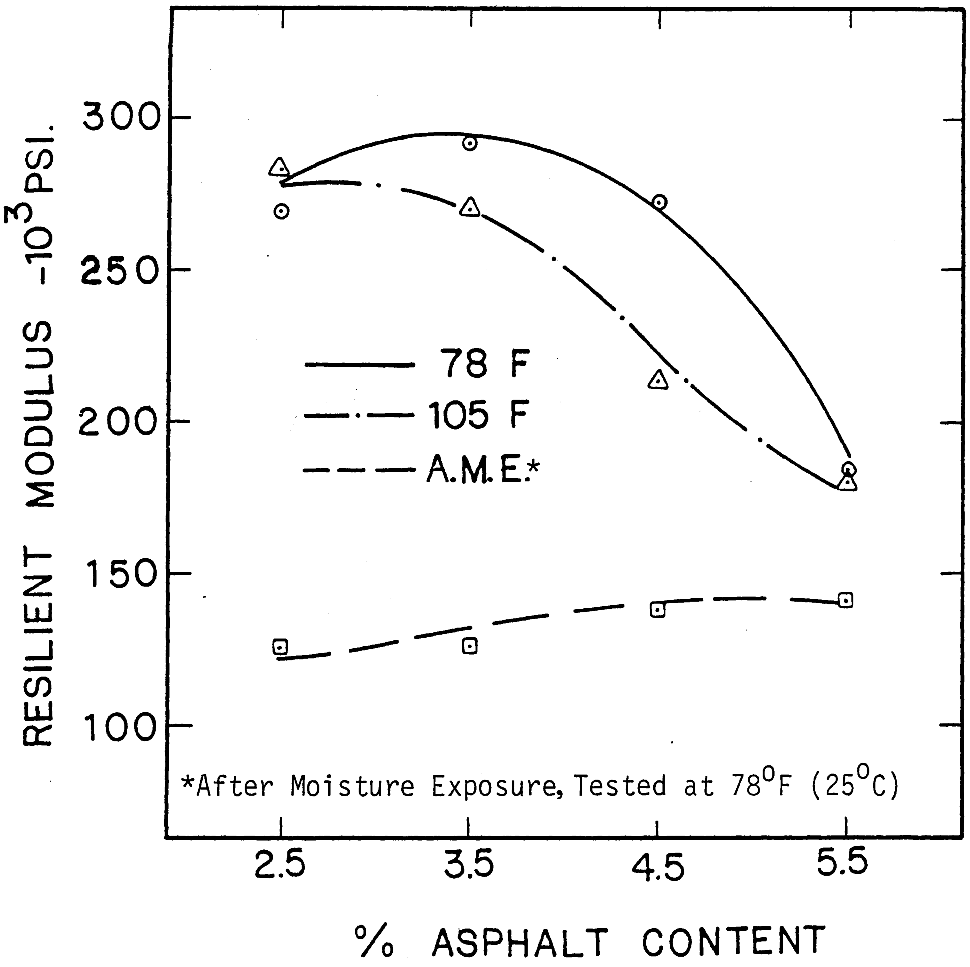


Figure 59. Resilient Modulus Versus Asphalt Content for Foamix (C-2)

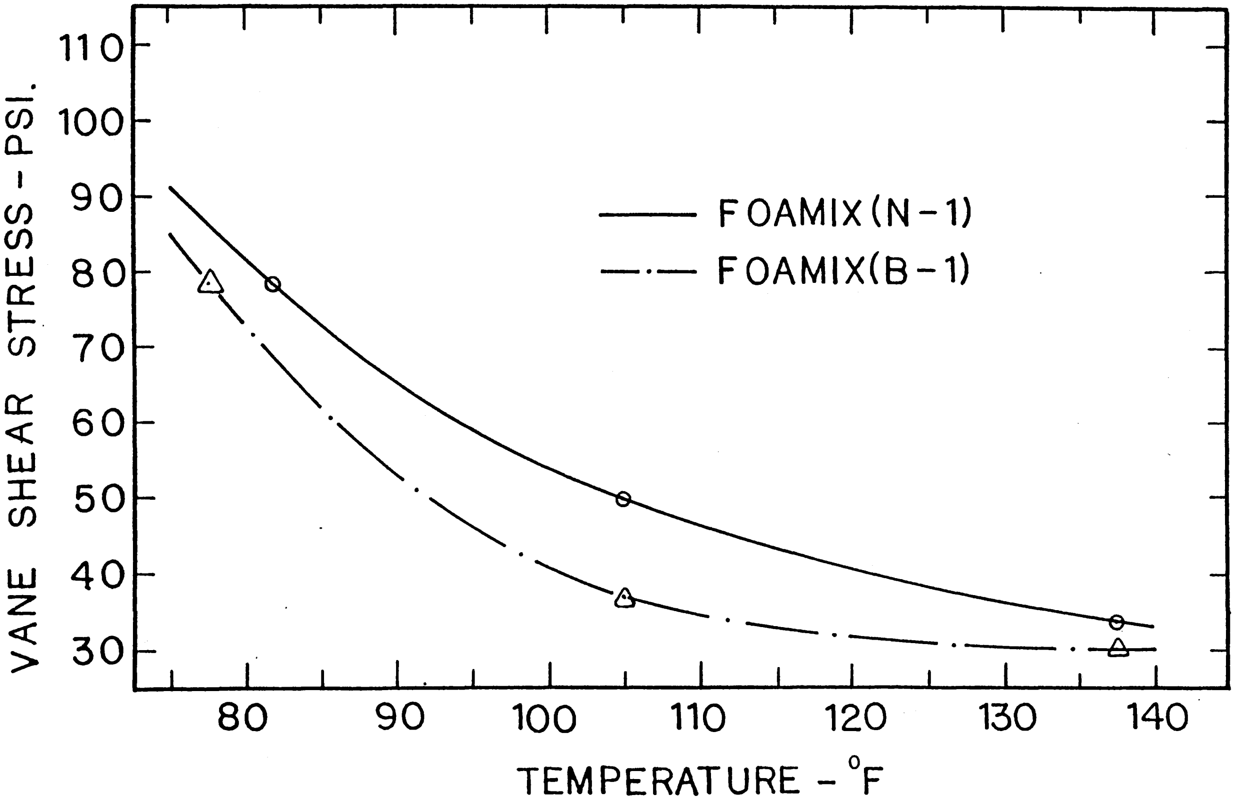


Figure 60. Effect of Temperature on the Vane Shear Strength of Foamixes (N-1) and (B-1) After Long Curing

VITA

2

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Candidate for the Degree of

Doctor of Philosophy

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