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THE EFFECTS OF MAXIMUM AGGREGATE SIZE ON PROPERTIES OF ASPHALT AGGREGATE MIXES

Sponsored by

National Stone Association
Washington, D.C.

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ABSTRACT

There are many factors which may affect the properties of asphalt concrete and one of these is the size of the largest aggregate used in the mix. This research project involved the analysis of the effect of varying the maximum aggregate size on the properties of an asphalt mixture. The aggregate in all mixes evaluated consisted of 100% crushed limestone.

The five different mix designs which were evaluated included aggregate having gradations that contained maximum aggregate sizes of 3/8, 1/2, 3/4, 1, and 1-1/2 inches. The asphalt content for all mixes was selected to provide an air voids content of four percent under a compactive effort in the Gyratory Testing Machine equivalent to 75 blows of a Marshall hammer.

All mixes produced with the five gradations were subjected to a testing program which included tests to evaluate Marshall stability and flow, indirect tensile strength, creep, and resilient modulus. Specimens for mix design and evaluation of mixture properties were compacted in a four inch diameter mold.

In addition, specimens at optimum asphalt content were prepared in a six inch diameter mold and were tested using the indirect tensile test and the creep test. These results were then compared to those from the four inch diameter specimens for the same aggregate gradations. The six inch diameter specimens were compacted to provide the same density as that measured for the four inch diameter specimens.

Test results indicated that mixes with larger aggregate designed with an air voids content of four percent were generally stronger than mixes prepared with smaller aggregate. The mixes with larger aggregate also required significantly less asphalt with no appreciable decrease in resistance to cracking as measured by tensile strain at failure.

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I. INTRODUCTION

Background

The effects of using large aggregate in asphalt mixes have been researched and speculated upon for many years. Patents were issued as early as 1903 for bituminous mixes which contained aggregate as large as three inches (8). Research is sparse, however, when one looks at a comparison of mixtures over a range of maximum aggregate sizes.

While large aggregate mixes have been used in specialized situations such as storage yards for equipment and materials (22), they are not currently used or accepted on a regular basis for highway mixes. The wide acceptance of the Marshall design procedure as well as the Hveem procedure may be a major factor limiting the use of large aggregate because standard 4 inch mold sizes and testing equipment limit aggregate maximum size to one inch. Production and placement of mixtures containing large aggregate in the field is also a problem and thus discourages the use of large aggregates.

Objectives

This study was conducted to determine the relationship between asphalt mixture properties and maximum aggregate size. An additional aspect of this study was to compare the differences in test results between four inch and six inch diameter specimens.

Scope

Testing procedures used in this project were chosen to analyze the effects of varying the size of the largest aggregate in a gradation. The tests used in this study included Marshall stability and flow, indirect tensile, static creep, and resilient modulus. All of the tests for this project were performed in the laboratory and all test specimens were prepared there. No specimens were taken from the field nor were any tests performed in the field. Also, no attempt was made to try to correlate laboratory test results to any conditions in the field.

Gradations were selected to contain 3/8, 1/2, 3/4, 1, and 1 1/2 inch maximum size aggregate. The aggregate was sampled so that all sizes came from the same location in the quarry and thus had the same properties. One sample of asphalt was used for all tests. Thus, every precaution was taken to insure that the test results focused on the effects of maximum aggregate size only and did not include the effects of varying the properties of materials.

Research Plan

The research plan for this project was designed to analyze the effects of changing maximum aggregate size on the properties of an asphalt mix. Tests were conducted to analyze Marshall stability and flow, indirect tensile strength, resilient modulus and creep. Six inch diameter specimens were prepared and analyzed for indirect tensile strength and creep and the results from the 4 inch and 6 inch diameter specimens were compared.

The five gradations used in this study were designed using a 0.45 power maximum density curve and were adjusted to meet Federal Highway Administration guidelines (11). This was done to more closely relate to actual use in the field.

All asphalt concrete specimens were prepared in this study using a Gyratory Testing Machine. However, the number of revolutions of the Gyratory Testing Machine was calibrated to produce a density equal to that achieved with 75 blows of the Marshall hammer.

The research data generated by the tests in this plan were organized so that trends could be identified. Analysis of these results was the final step in determining the effects of maximum aggregate size on the properties of asphalt-aggregate mixtures.

II. REVIEW OF LITERATURE

Causes of Rutting

Modern traffic levels and tire pressures have resulted in increased stress on modern pavements. Brown (4), in a paper presented at an AASHTO/FHWA Symposium in Austin, Texas, in 1987, listed several conditions which may be aggravated by these stresses and which may result in rutting.

The potential problems causing rutting failure listed by Brown included excessive asphalt content caused by improper laboratory procedures, excessive use of natural sand or minus #200 material, improperly crushed aggregate, maximum size coarse aggregate that was too small, and density obtained in the field that was too low (4).

A study of rutting in Canada by Huber and Heiman (13) analyzed the condition of asphalt concrete as it was designed, after it was constructed, and as it existed at the time of their study. They used cores from between the wheelpaths to represent conditions immediately after construction. The condition after traffic was represented by cores taken from the outer wheelpath and the characteristics of the mixes as they were designed were obtained from historical data and from construction records.

Huber and Heiman used regression analysis and threshold analysis to identify characteristic values which separated acceptable and unacceptable behavior. Among their findings, the threshold air voids content was 4% minimum. The threshold value for voids in the mineral aggregate (VMA) was 13.5% minimum and the voids filled threshold value was approximately 70% maximum. An analysis of the fractured faces proved difficult, but the acceptable value which Huber and Heiman eventually determined was 60 percent minimum. They did not specifically define fractured faces. The Marshall stability test was shown to be a poor indicator of rutting because tests conducted on mixes from rutted and non-rutted asphalt pavements yielded approximately the same stability values. Hveem stability correlated reasonably well with rutting and indicated a threshold value of 37 minimum. The threshold asphalt content was determined to be 5.1 percent maximum (13).

Huber and Heiman concluded that rutting resistance could not be separately related to traffic level or mix properties of the asphalt mixes. When rutting was analyzed according to deformation per number of single axle loadings, however, Huber and Heiman found a strong correlation with air voids, voids filled, asphalt content, and Hveem stability. Performance was directly affected if voids filled were greater than 70%, air voids were less than 4%, or asphalt content was greater than 5.1%. They found that fractured faces, VMA, and Hveem stability seemed secondary and Marshall stability, flow, penetration, and viscosity showed little correlation to rutting resistance (13).

A British study of roadway bituminous base material by Brown and Cooper (6) used various gradations with maximum aggregate size up to 40 mm (1.57 inch) to analyze elastic stiffness, fatigue life, and rutting resistance. They used four full scale field trials and laboratory work in this study. Testing methods included a repeated load triaxial test, triaxial creep, uniaxial creep and Marshall stability.

The creep results obtained by Brown and Cooper indicated that asphalt mixes prepared with 100 and 200 penetration grade asphalt showed no significant difference in permanent deformation. Aggregate gradation, however, had a significant effect on permanent deformation. Mixes with dense graded and gap graded aggregates were compared and the gap graded mix experienced significantly more permanent deformation than the dense graded mix (6).

Brown and Cooper's Marshall stability results led to inconsistent conclusions. In one case, Marshall stability gave indications that were opposite those of the triaxial test. They concluded that the inconsistencies were caused by the fact that they were using aggregate larger than that specified in the Marshall procedure (6).

Effects of Coarse Aggregate

In a 1986 ASTM paper, Brown, McRae and Crawley (5) presented results which implied the advantages of larger aggregate while not analyzing larger aggregate specifically. Their test results showed that both stability and tensile strength decreased as voids in the mineral aggregate (VMA) increased. Since VMA is generally higher for smaller aggregate, stability and tensile strength decreased as aggregate size decreased.

Other advantages to using large aggregate which were discussed by Brown, McRae and Crawley included improved skid resistance and lower optimum asphalt content. They did mention, however, that the Mississippi State Highway Department had reduced the maximum aggregate size for its surface mix specifications from 1/2 inch to 3/8 inch because crushing to the 1/2 inch size produced some elongated aggregate which had poor skid resistance (5).

The effects of using aggregate up to 2 1/2 inches in size were investigated by Khalifa and Herrin (17). Their study covered two broad areas. First, they analyzed the effects of aggregate size on the physical properties of the mix such as air voids, density, and voids in the mineral aggregate. Next, they analyzed the effects of using larger aggregate on the ability of construction equipment to place the asphalt concrete and the cost of producing the asphalt mixture.

The general conclusions by Khalifa and Herrin were that unit weight increased as aggregate size increased and VMA and air voids decreased with increased aggregate size for any given asphalt content tested. Mixture strengths were determined using triaxial compression at a constant rate of deformation and three different lateral pressures.

The triaxial test results indicated that for the same asphalt content and lateral pressure, the strength of the mixes tended to decrease with increased aggregate size. However, they also concluded that high strength for large aggregate mixes was possible but at a much lower asphalt content than for conventional mixes.

A laboratory and field study published by the National Asphalt Pavement Association (NAPA) gave the results (among many results) of two mixes (1). One had a maximum aggregate size of 1/2 inch and the other a maximum aggregate size of 1 1/2 inches. Among other points, the report described the problems of preparing laboratory mixes with the currently available 4 inch diameter molds. A modified Marshall procedure was used in compacting samples in four inch diameter molds and samples were compacted in six inch diameter molds using a vibrating hammer. Table 1 gives some of the results of this study. The large stone mix in Table 1 consisted of 50% railway ballast and 40% crushed graded gravel. The report did not say specifically but the conventional mix was probably crushed gravel.

The most obvious point made in Table 1 was the improvement in stability for larger maximum aggregate size. Another point, however, was that the film thickness remained basically the same between the two mixes even though the asphalt content for the larger mix was significantly lower. The film thickness was the same because the larger maximum size gradation had a smaller aggregate surface area (1).

It is important to notice some degree of inconsistency in Table 1. Examination of the gradation curves included in the NAPA report showed that there was a significant difference between the two gradations in regards to the amount of material in the sand sizes. The conventional mix appeared to have contained approximately 40%

Table 1

Comparison Between Characteristics of Large Stone Mix and Conventional Mix

	Large Stone Mix		Conventional Mix
Gradation	1 1/2" nominal max. size, stone-filled		1/2" nominal max. size, dense graded
Design Asph. Conc.	3.5%		5.2%
A.C. Grade	AC-20		120/150 pen
Stability (lbs.)	2746 (1)		1225
Flow (0.01 in.)	7.0 (1)		8.0
Voids (%)	3.3 (1) 5.2 (2)		4.0
VMA (%)	10.5 (1) 12.3 (2)	11.9 (3) 13.6 (3)	16.3 (3)
Film Thickness (microns)	8.7		8.2

(1) Modified 4" Marshall Procedure
(2) 6" Diameter Vibratory Compacted Specimens
(3) Based on effective specific gravity

(Acott, Holt, and Puzinauskas, 1988)

natural sand and the large stone mix appeared to contain only about 20%. The amount of natural sand in the two mixes was estimated from the shape of the two gradation curves. This much natural sand (40%) could have a very detrimental effect on the strength of the conventional mix. Also, the difference in the asphalt used in the two mixes could be detrimental to the stability of the 1/2 inch aggregate mix. AASHTO M 226-80 indicates that an AC-20 asphalt cement has a minimum penetration of 40. The asphalt cement in the 1/2 inch aggregate mix had a penetration of 120/150, which is much less viscous (approximately AC-5 according to AASHTO M 226-80) than the AC-20 and would put the 1/2 inch mix at a disadvantage.

The U.S. Army Engineer Waterways Experiment Station conducted a study for the Air Force which analyzed the effects on asphalt concrete pavement performance of increasing the maximum aggregate size in the mix from 3/4 inch to 1 inch (20). This study was conducted to develop mixes to withstand the tire pressures of modern fighter aircraft, some of which reach 350 to 400 psi. The study included evaluations of tensile strength, unconfined creep, aging, and direct shear. Factors that were evaluated included compactive effort and asphalt viscosity.

The investigators concluded that the level of compactive effort did not significantly affect durability (over the range of compactive efforts studied) but that the asphalt content did. Varying the compactive effort over the ranges studied had little effect on the voids in the total mix. Higher asphalt contents meant lower voids and produced a mix that was less subject to aging. Creep resistance was best with the higher compactive efforts when they were performed on specimens that were mixed with an asphalt content that was slightly lean of optimum. The highest compactive efforts also produced the greatest shear strength. The 1 inch mixes performed better at the higher compactive efforts than did the 3/4 inch mixes and AC 40 asphalt produced mixes that were stronger than mixes produced with AC 20.

The ASTM procedure for preparing 4 inch diameter specimens using the Marshall hammer recommends that it be used for aggregate smaller than one inch. Cross (7) studied the effects of maximum aggregate size on specimens of asphalt stabilized base material prepared in 4 inch molds.

Cross characterized the limestone mixes according to those with maximum aggregate size greater than 1 inch and those less than 1 inch. His test results indicated that the plus 1 inch aggregate yielded a higher stability but that the stability values for the plus 1 inch material were "very erratic." The larger aggregate also required a slightly higher optimum asphalt content. This optimum asphalt content was the opposite of what was expected because the larger aggregate should have required less asphalt to maintain the same voids.

Khalifa and Herrin (17) used maximum sized aggregate ranging from 3/4 inch to 2 1/2 inches. They did not use standard molds for sample fabrication for the material exceeding 1 inch in size. Instead, they prepared large slabs of asphalt concrete and cored the necessary specimens from the slab. They listed several advantages to this method. They said it allowed for the best possible conditions in producing identical specimens, it produced samples that were more representative of field conditions, it avoided human and environmental errors which may occur in making individually molded specimens, it circumvented the ASTM ratio rule of 4 to 1 in determining minimum specimen diameter from aggregate size, time was saved, and the distribution and orientation of the aggregate in the mix could be examined.

Brown and Cooper (6) used a method similar to Khalifa and Herrin to prepare their specimens. They used a large mold constructed from 120 mm (4.72 inches) square steel box sections. The box sections were used as the outer walls of the mold and were stacked two high (overall mold dimensions were not given). Thus, the asphalt concrete was placed in two layers that were each approximately 4.72 inches thick. The mold was capable of holing 1 1/2 to 2 tons of mix and could facilitate the use of large equipment. Samples were then cored from the mold in diameters of 100 mm (3.94 inches) both vertically and horizontally by removing blocks of material from the molded slab.

Kandhal (15) has reviewed the effects of preparing 6 inch diameter specimens using a Marshall procedure adapted from the 4 inch diameter procedure. In order to produce the same amount of energy per unit volume in the 6 inch specimens as in the 4 inch, a 22.5 lb. hammer was recommended over the standard 10 lb. hammer. Drop height remained the same but the number of blows was increased by 50 percent. Some crushing of the surface aggregate was observed but Kandhal did not believe it was sufficient to affect the Marshall properties.

Effects of Fine Aggregate

A result of research by Kalcheff and Tunnicliff (14) in 1982 demonstrated the effects of filler material on mix properties. They found that for a given aggregate, optimum asphalt content was higher for aggregate containing less filler material (material passing a #200 sieve) and lower for aggregate containing more filler material.

Two of the gradations they tested had a 1/2 inch maximum aggregate size and were very similar except in regard to fine material. The tensile strength increased significantly when filler material was increased in mixture B from mixture A (Table 2). The

Table 2

Change in Indirect Tensile Strength from Addition of Filler Material

Mixture Designation	A	B
Fine Aggregate	Tensile Strength, p.s.i.	
Natural Sand	132	166
VA Limestone	148	169
Diabase	134	156

(Kalcheff and Tunnicliff, 1982)

gradations were similar but mixture A contained 5.5% minus #200 material and mixture B contained 9.5%.

A study of the effects of the properties of various types of aggregates and gradations using one type of asphalt cement was accomplished by Evans and Lott (10) while working for the Amoco Oil Company. They used a test track facility to study these aggregate and gradation variables on pavement flow deformation. Test traffic conditions were set at 91 psi tire pressure, 1000 lb. wheel load, and 21 mph. They determined that the primary factors affecting flow deformation were asphalt content and pavement temperature. Secondary factors were the amount of fines in the mix and the aggregate gradation.

Wedding and Gaynor (23) studied the effects of varying the amount of sand (defined by them as material passing the #8 sieve) in combination with the use of crushed coarse aggregate. They used Marshall compaction procedures and stability testing to analyze the effects of varying the amount of crushed material in a mix. The optimum asphalt content for each mix tested was determined by using the average of the asphalt contents that provided the peak of the stability curve, 4% voids, the peak of the unit weight curve, and 80% voids filled. The percentage of crushed material in the aggregate was varied from 0, 50, 75, and 100% in the coarse aggregate and the percent sand was varied from 25, 35, and 45% (percent by weight of total mix). They analyzed both natural and crushed sand at these three percentages. Natural sand and crushed sand were used for each of the three and contents. The crushed aggregate was quartz gravel from Maryland which was crushed in a small jaw crusher and had at least 2 fractured faces.

Generally, Wedding and Gaynor found that for the gradations which had 75% and 100% crushed coarse material, the optimum crushed sand content was 35%. A 35% crushed sand content yielded the maximum Marshall stability, the minimum optimum asphalt content, the minimum VMA, and the maximum unit weight. The 50% and 0% crushed coarse material yielded an optimum crushed sand content of 45%. For the natural sand, the 35% sand content was the optimum for the gradations having 100, 75 and 50% crushed coarse material. Only the 0% crushed coarse material exhibited a change in optimum sand content (natural sand) from 35 to 45% (23).

Anderson and Tarris (2) studied baghouse dust which had been collected from 26 plants in 11 states and included 5 different types of aggregate. They found that variability in baghouse efficiency produced gradations with varying coarseness. Some baghouse material may act as mineral filler in a mixture but some may act as fine sand. They also found that the stiffness of an asphalt mixture was not uniquely related to the fineness of the dust but that in most cases, "the greatest stiffening was produced by one-sized, finer dust."

Effects of Film Thickness

The strength characteristics of asphalt films were analyzed by Marek and Herrin in 1968 (19). Their analysis did not include mixing and testing the asphalt cement as a part of an asphalt concrete mix but rather as a thin film sandwich between two test blocks. Among the variables in the test were temperature, deformation rate, film thickness, consistency and source of the asphalt cement.

Haas, in a discussion of the Marek and Herrin article, presented tensile strength results for two asphalts from different sources in Figure 1. This evidence clearly indicates an optimum film thickness with respect to tensile strength. Besides indicating tensile strengths which varied according to source, Marek and Herrin said that they also varied from the same source according to asphaltene content. The higher asphaltene content usually had the higher tensile strength. This was not necessarily true, however, for asphalt cements from different courses (19).

Creep Testing

Van de Loo (21) analyzed the relationship between rutting and creep testing. He analyzed data from static and dynamic loads on a test track and static and dynamic creep tests.

He found that the stiffness of the mix decreased as the number of load applications increased. When compared at equal asphalt viscosity, the dynamic stiffness modulus of a mix was always higher than the static stiffness modulus. After analyzing the use of results from laboratory prepared specimens to predict rutting behavior, Van de Loo concluded, "It may be that the main purpose of laboratory test methods must be limited to the ranking of materials rather than the prediction of rut depths" (21).

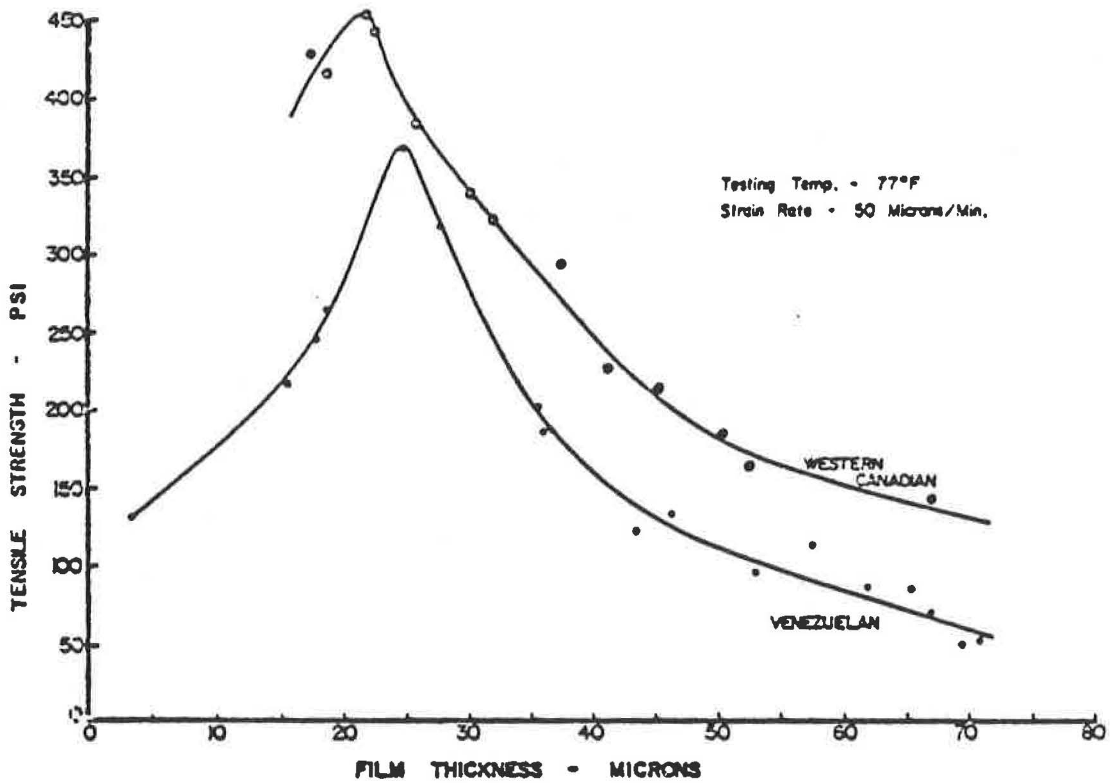

Figure 1. Tensile Strength vs. Film Thickness Relationship for Two Unaged Asphalts (Viscosity at 140 deg. F. approximately equal to 1200 Poises)

(Marek and Herrin, 1968)

Van de Loo found that a good correlation between creep tests and rutting behavior could be accomplished only if the creep tests were carried out at a "sufficiently low" stress level, and if an experimentally determined correction factor was used in the prediction process to allow for any "dynamic effects." This correction factor was derived from repeated load creep tests and then simply multiplied by the creep test strain in order to adjust the strain to the expected rut depth. Van de Loo proposed the following equation for predicting rut depth:

$$\text{rut depth} = C \times H \times e \qquad (21),$$

where C = correction factor,
H = pavement thickness, and
e = strain.

Another study using creep tests to construct a model for predicting rutting was done by Lai and Hufferd (18). Their basic premise was that since asphalt is not a linear visco-elastic material even at small stresses, then creep recovery cannot be predicted using traditional linear visco-elastic theory.

They divided creep strains into two parts, those that were recoverable and those that were not. The model represented the recoverable strains with a Kelvin chain and the irrecoverable portion with a non-linear dashpot. Creep tests were run on samples prepared in the laboratory. The researchers claimed better success in predicting deformation using their model than by using traditional linear visco-elastic theory. Their equations, however, contained empirical constants which may have limited the accuracy of their modeling technique for widespread use.

Indirect Tensile Test

Kennedy (16) has analyzed the indirect tensile test and its use in determining many aspects of asphalt concrete performance. Based on both static and dynamic loading, Kennedy concluded that the indirect tensile test may provide information on fatigue, elastic modulus, Poisson's ratio, and permanent deformation. His conclusions regarding permanent deformation were based on his work and that of others.

An interesting result of Kennedy's research was the variability of the Poisson's ratio. For static loadings, the majority of values ranged from 0.08 to 0.36, while the majority of instantaneous resilient Poisson's ratios (ratios derived from repeated loadings) ranged from 0.10 to 0.70. A Poisson's ratio of 0.50 indicates no volume change in the test specimen. Values greater than 0.50 indicate an increase in volume and thus may be suspect. Kennedy, however, indicated that values greater than 0.50 were often achieved after a "relatively large number of load applications." Thus, the repeated loading produced strain in the horizontal direction (the direction of stress that causes a tensile failure along a vertical plane) larger than the strains in the vertical direction (direction of loading) as the specimens approached fatigue failure. The ratio increased with increased load applications with a rapid increase at about 70 to 80 percent of fatigue life (16).

III. SAMPLE PREPARATION, TEST PROCEDURES AND RESULTS

Tests were selected to evaluate those properties of asphalt-aggregate mixtures that could be correlated with performance. A copy of the overall test plan to determine these properties is provided in Figure 2. A complete summary of all data is provided in the Appendix.

Determination of Aggregate Gradation

The aggregate used in this study was 100 percent crushed limestone from the quarry of Vulcan Materials in Calera, Alabama. The gradation specifications for each maximum size aggregate were those of the Federal Highway Administration (FHWA) and are shown in Table 3 (11).

The specific percentages passing each sieve size were determined using a theoretical maximum density (or 0.45 power) curve first derived by Nijboer in 1948 (6) from the test results of many gradations to determine a gradation to maximize density. The gradation determined to produce the maximum density was,

$$P = 100\,(S/M)^{0.45},$$

where P = percentage passing any particular sieve size,
S = the size of opening for that sieve, and
M = the maximum aggregate size in the gradation.

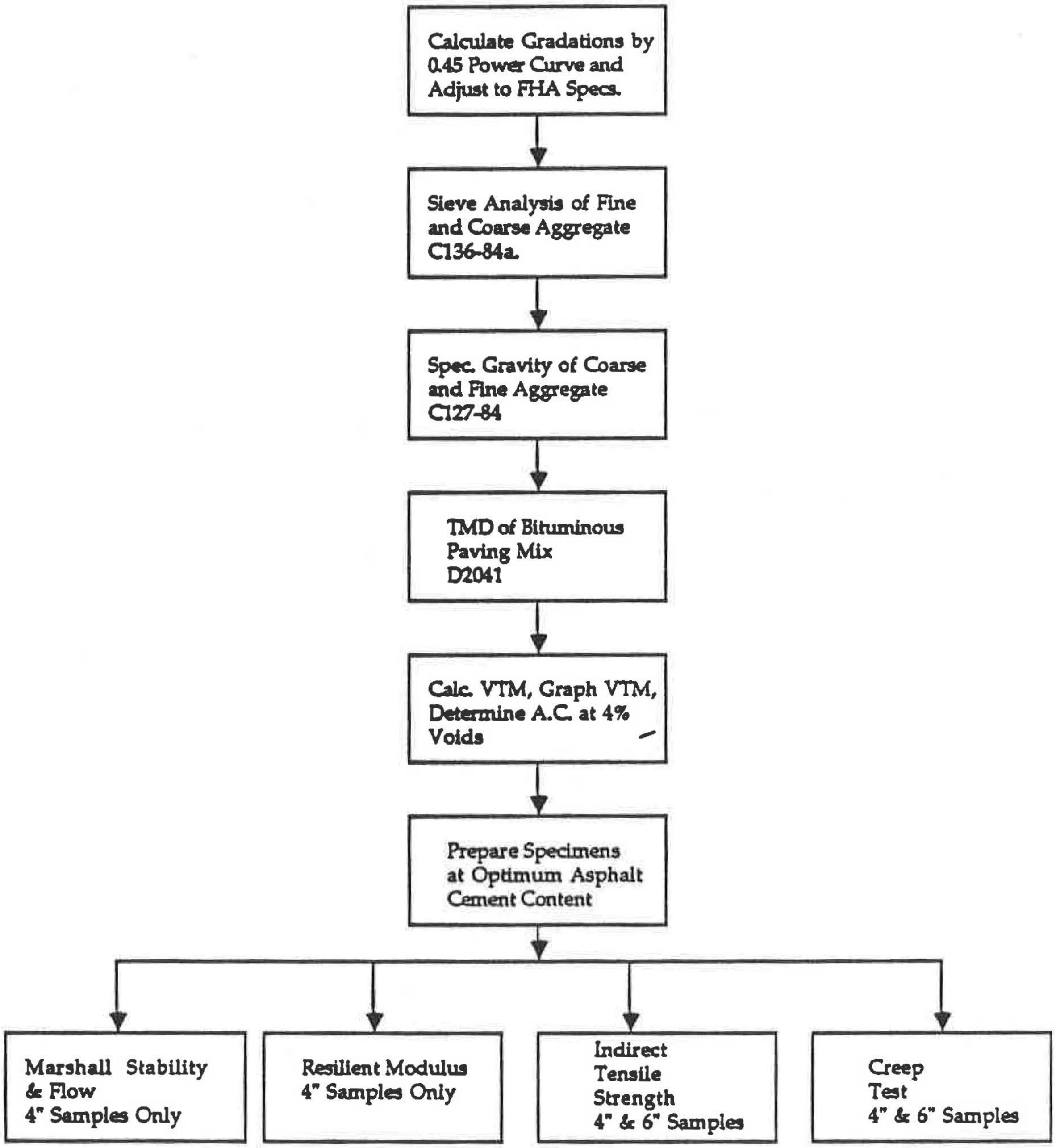

Figure 2. Test Plan

Table 3

Gradation Ranges for Asphalt Concrete Mixes

Sieve Designation	Grading Designation A	B	C	D	E
2 inch	100	-	-	-	-
1 1/2 inch	97-100	100	-	-	-
1 inch	-	97-100	100	-	-
3/4 inch	66-80	-	97-100	100	-
1/2 inch	-	-	76-88	97-100	-
3/8 inch	48-60	53-70	-	-	100
No. 4	33-45	40-52	49-59	57-69	97-100
No. 8	25-33	25-39	36-45	41-49	62-81
No. 40	9-17	10-19	14-22	14-22	22-37
No. 200	3-8	3-8	3-7	3-8	7-16

The calculated gradations were compared to the FHWA specifications. The 1 1/2 inch gradation used Grading Designation"A" (Table 3), the 1 inch used "B", the 3/4 inch used "C", the 1/2 inch used "D", and the 3/8 inch was interpolated between Grading Designations "D" and "E". All the gradations except the one with 1 1/2 inch maximum size aggregate had to be adjusted at the #200 sieve size to fit the FHWA specification envelope. That is, the amount of material passing the #200 sieve had to be reduced. The final gradations are shown in Figure 3 and Table 4.

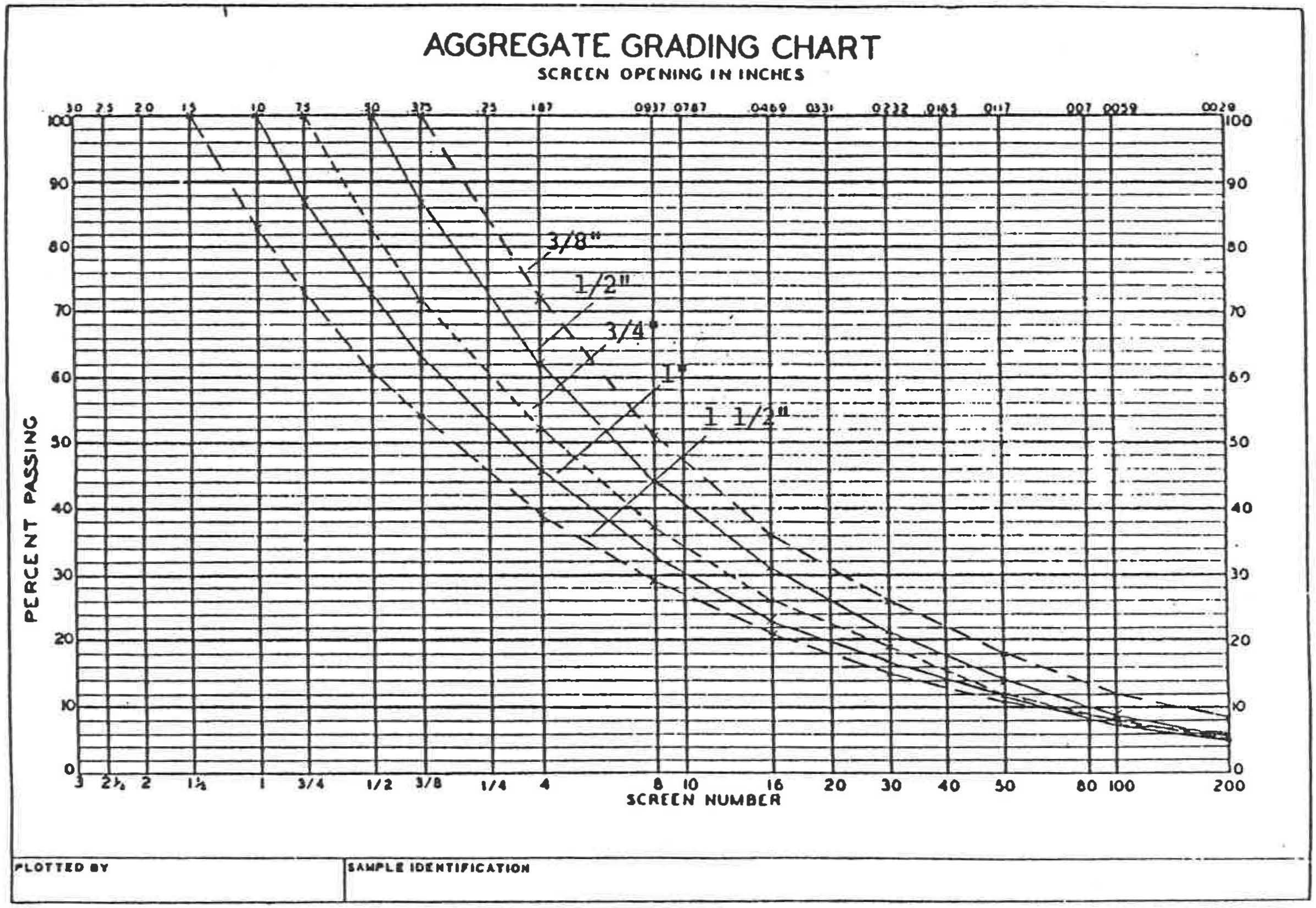

Figure 3. Test Gradations

Table 4

Mix Gradations Arranged by Maximum Aggregate Size

Sieve	3/8 in.	1/2 in.	3/4 in.	1 in.	1 1/2 in.
1 1/2"					100
1"				100	83
3/4"			100	87	73
1/2"		100	83	73	61
3/8"	100	87	72	63	54
#4	72	62	52	46	39
#8	51	44	37	33	29
#16	36	31	26	23	21
#30	26	21	19	17	15
#50	18	14	12	12	11
#100	12	9	8	8	8
#200	8.2	5.8	5.2	5.5	6.1

Properties of the Asphalt Cement

The AC 20 asphalt cement used in this study was produced by the Mobile, Alabama, refinery of Chevron Corporation. Its specific gravity was 1.032 and pen was 82 at 77 deg. F. Viscosity testing indicated 1940 Poises at 140 deg. F. and 403 Cst at 275 deg. F. A Cleveland Open Cup flash test indicated a flash point of 555 deg. F.

Aggregate Specific Gravity Determination

The crushed limestone was split into five sizes for specific gravity determination. These sizes were chosen so that their test results could be easily and quickly related to their respective sizes in the test gradations. The five sizes were 1 1/2 - 3/4 inch, 3/4 - 3/8 inch, 3/8 - #8, #8 - #30, and minus #30.

The specific gravity of the aggregate was determined for the five different aggregate size groups. Test method ASTM C 127-84 was used to measure the specific gravity and absorption of the aggregate larger than the #8 sieve and test method ASTM C 128-84 was used to measure the specific gravity and absorption of the material passing the #8 sieve. The specific gravity results are shown in Table 5.

Table 5

Specific Gravity and Absorption of Aggregates

	Aggregate Size				
	1 1/2-3/4	3/4-3/8	3/8-#8	#8-#30	Minus #30
Bulk Sp. Gr.	2.739	2.782	2.777	2.783	2.754
Bulk Sat. Sur. Dry Sp. Gr.	2.744	2.789	2.785	2.801	2.786
Apparent Sp. Gr.	2.753	2.801	2.800	2.835	2.843
Absorption (%)	0.177	0.237	0.290	0.664	1.133

Separation of Aggregate for Blending

The aggregate was dried and then separated into individual sizes by dry sieving. Sufficient material of each aggregate size was sieved and stored in an amount sufficient to prepare all the required specimens. Material was separated on all sieve sizes shown in Table 4 so that all specimens could be closely controlled during preparation.

In order to insure that all blends contained the correct amount of dust (minus #200), a representative sample was taken from each aggregate size to determine its content of minus #200 material. A mechanical splitter was used to split a sample from each aggregate size into the desired amount for measuring aggregate gradations as specified by ASTM D 75-82.

The amount of minus #200 material that was contained in each separated aggregate size was determined by running a washed gradation. Sufficient minus #200 material was measured on the #50-#100 an #100-#200 size material to require that modification be made during blending to account for the retained minus #200 material.

Compaction Calibration

The number of revolutions of the gyratory testing machine (GTM) was selected to produce a density equal to that produced by a 75 blow compactive effort using the Marshall procedure. Three specimens were prepared using the Marshall hammer (75 blows) and their specific gravity was averaged. Three specimens were then prepared for intervals of 10 revolutions of the GTM (10 through 60) and their specific gravities were averaged for each number of revolutions. The mixtures used for calibration contained 1/2 inch maximum aggregate size and this calibration was used for all mixes.

This procedure showed that approximately 30 revolutions at a pressure of 200 psi and and one degree gyratory angle produced a density equal to that obtained with a 75 blow compactive effort (Table 6 and Figure 4).

Table 6

Gyratory Calibration Data

Average Specific Gravity - Marshall	2.534
Average Specific Gravity - Gyratory	
10 revolutions	2.445
20 revolutions	2.493
30 revolutions	2.536
40 revolutions	2.548
50 revolutions	2.555
60 revolutions	2.558

All these specimens were 4 inches in diameter.

All specimens tested in this calibration procedure were produced with a 1/2 inch maximum size gradation. This size gradation was selected because four inch diameter specimens were used in the calibration process and 1/2 inch aggregate would not produce any interference problems from the sides of the mold.

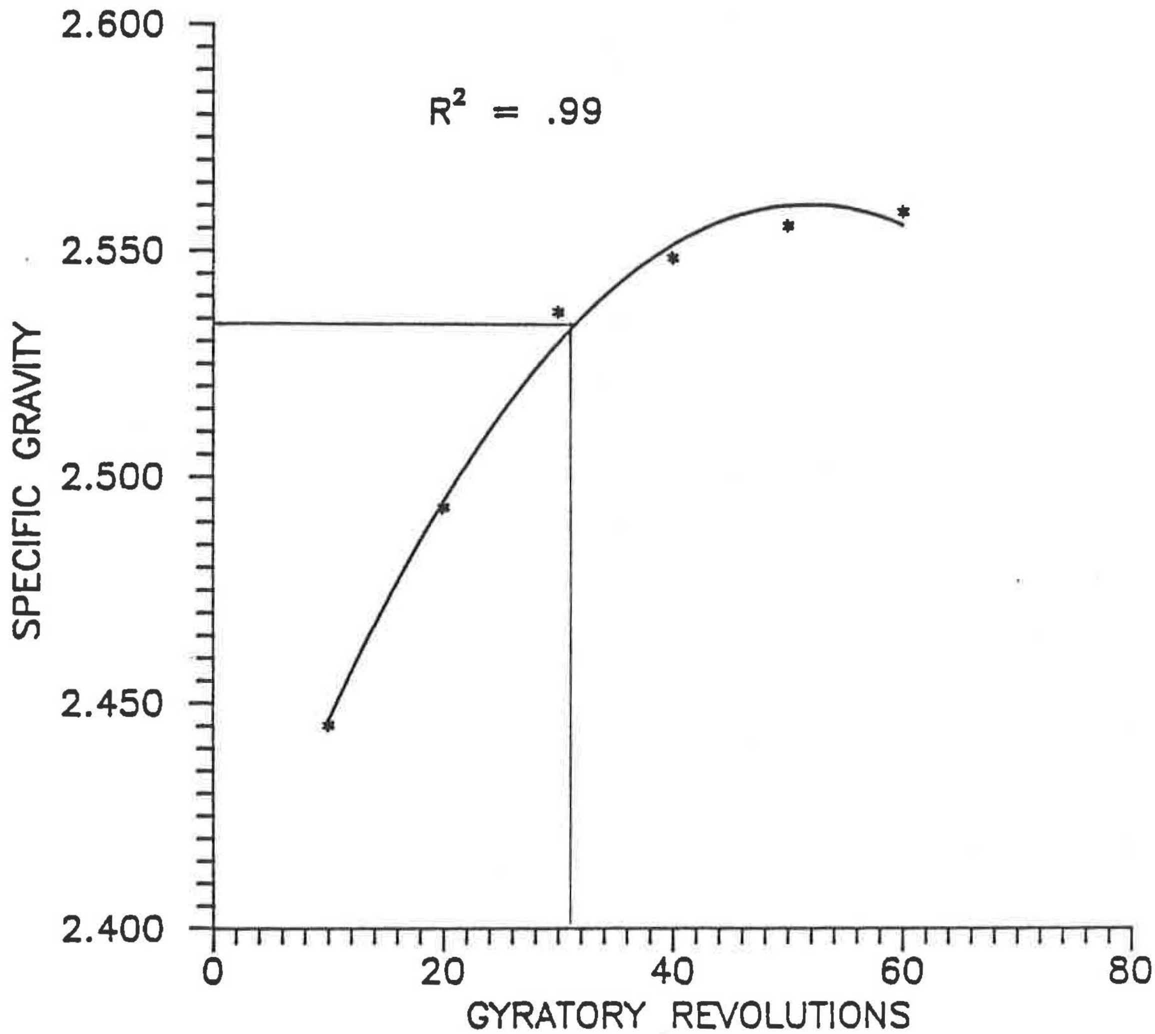

Figure 4. Gyratory Calibration Curve for 1 deg. Gyratory Angle 200 psi, and 1/2" Maximum Size Aggregate

Six inch diameter specimens were also prepared using 30 revolutions, 200 psi, and 1 degree angle. In order to determine if this produced the same specific gravity in the 6 inch diameter specimens as it did in the 4 inch diameter specimens, the specific gravity for the 6 inch diameter specimens containing 1/2 inch maximum size aggregate was compared to the specific gravity of the 4 inch diameter specimens tested for creep and indirect tensile strength (because these were the tests conducted on the 6 inch specimens). The 4 inch diameter specimens yielded an average specific gravity of 2.505 and the 6 inch specimens yielded an average specific gravity of 2.501. This verified that the specific gravity values were very close for the 2 specimen sizes, hence the compactive effort selected for the 6 inch diameter specimens was considered satisfactory.

Mix Design and Specimen Preparation

The specimens to be tested were prepared at the asphalt content (optimum) necessary to produce 4% air voids. Thus, for each gradation the optimum asphalt content was determined by preparing three specimens at asphalt contents of 3.5, 4.0, 4.5, 5.0, 5.5 and 6.0%. The mixes containing 1 and 1 1/2 inch maximum aggregate sizes required additional specimens be made at 3.0% asphalt content. The percent air voids were calculated using the average bulk specific gravity of the specimens at each asphalt content and the theoretical maximum specific gravity (Rice) of one of the specimens at the same asphalt content. Bulk specific gravity was determined using ASTM D

2726-86 and the theoretical maximum specific gravity was determined using ASTM D 204-178, using a Type "A" bowl procedure.

A two minute mixing time was used on all mixes but the larger aggregate sometimes required additional mixing by hand after machine mixing in order to obtain a uniform coating of asphalt. The fine material which remained attached to the mixing bowl after mixing was always scraped from the bowl and added to the mix in the mold before the specimen was compacted.

All specimens were prepared using the procedure from the Marshall method described in ASTM D 1559-82. Aggregates were blended to total 1200 grams and the asphalt content was calculated as a percentage by weight of the total mix. The six inch specimens were prepared similarly using a total aggregate weight of 4050 grams. The six inch specimens were compacted with the gyratory machine using thirty revolutions, a pressure of 200 psi, and 1 degree angle to provide the same density as the 4 inch diameter specimens. Six inch specimens were not used in the mix design process but were produced at the determined asphalt content for the 4 inch diameter specimens to evaluate creep and tensile strength.

The results of these tests on 4 inch diameter specimens were plotted and the asphalt content that provided 4% air voids was selected from these curves to be the optimum asphalt content. The curves are shown collectively in Figure 5 and the optimum asphalt contents determined from these curves are shown in Table 7.

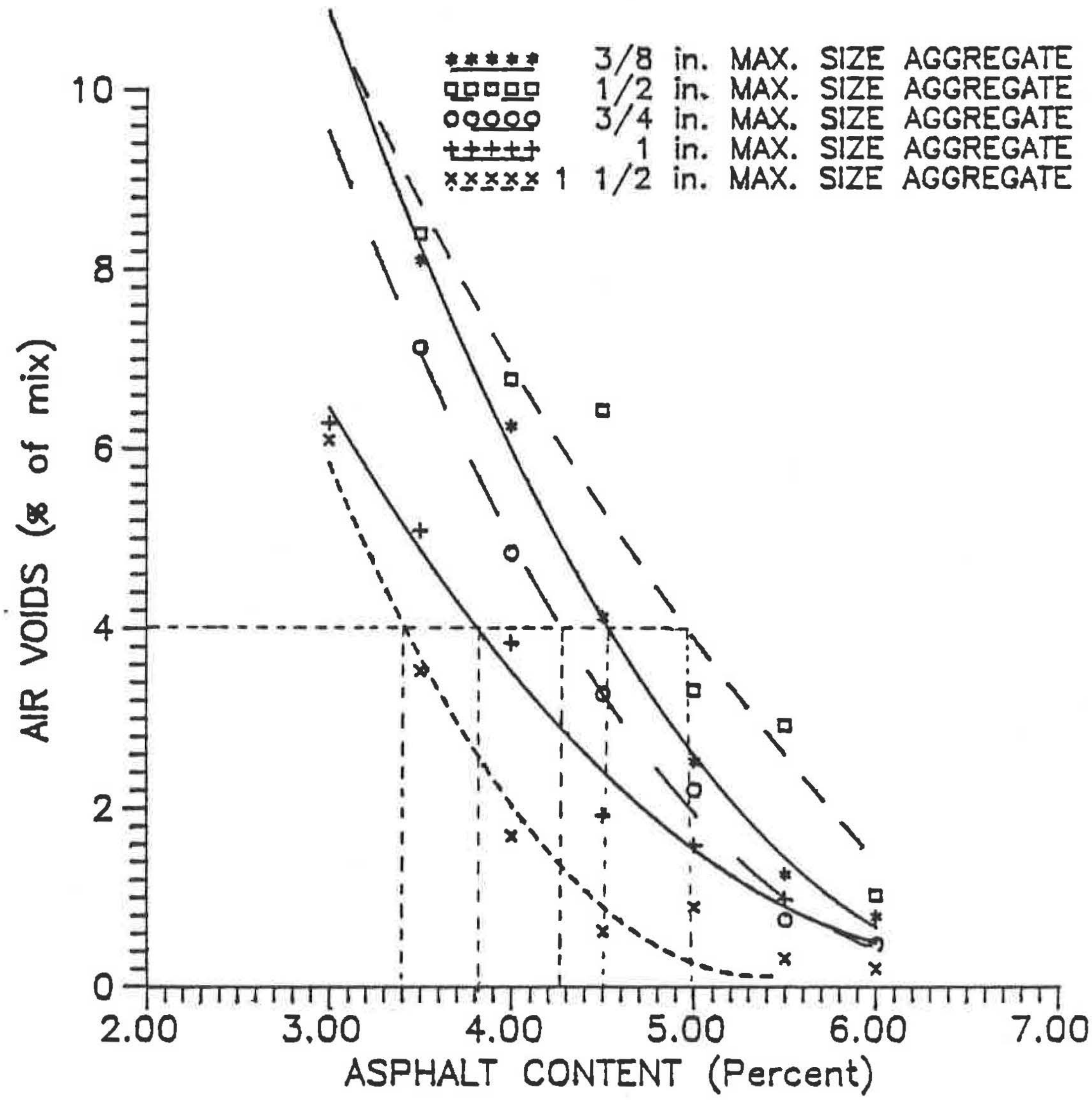

Figure 5. Mix Design Curves

Table 7

Asphalt Content for Each Gradation

Maximum Size Gradation	Asphalt Content
3/8 inch	4.5%
1/2 inch	5.0%
3/4 inch	4.3%
1 inch	3.8%
1 1/2 inch	3.4%

The equation used for determining the voids in the asphalt mixtures was,

$$\% \text{ VOIDS} = (1 = (\text{BULK SP. GR./TMD})) \times 100.$$

Testing

Marshall Stability and Flow Test

The Marshall stability and flow tests were conducted following the procedures described in ASTM D 1559-82 on three 4 inch diameter asphalt specimens of each gradation. The specimens were heated to 140 deg. F..in a water bath prior to measuring stability and flow. The Marshall stability and flow results are shown in Table 8.

Indirect Tensile Test

The specimens (both six inch and four inch) for the indirect tensile test were prepared as outlined above. This test was conducted following the procedure described in ASTM D 4128-82 at a

Table 8

Marshall Stability and Flow Results Using 4 Inch Diameter Specimens

Max. Agg. Size (in.)	Asp. Con.	Bulk Spec. Grav.	Stability	Flow
3/8	4.5	2.471	2275	13.0
3/8	4.5	2.492	2450	13.0
3/8	4.5	2.479	2450	12.0
Avg.			2392	12.7
1/2	5.0	2.465	2000	13.0
1/2	5.0	2.480	2025	12.0
1/2	5.0	2.509	2365	13.0
Avg.			2130	12.7
3/4	4.3	2.473	1820	12.0
3/4	4.3	2.516	2150	13.0
3/4	4.3	2.505	2162	15.0
Avg.			2044	13.3
1	3.8	2.526	2088	13.0
1	3.8	2.532	2513	14.5
1	3.8	2.530	2188	13.0
Avg.			2263	13.5
1 1/2	3.4	2.535	2000	14.5
1 1/2	3.4	2.531	2075	16.0
1 1/2	3.4	2.549	2626	15.5
Avg.			2234	15.3

temperature of 77 degrees F. and a standard load rate of 2 inches per minute. Three specimens were prepared and tested for each gradation in order to obtain an average indirect tensile strength for the gradation.

The loading heads for the six inch diameter specimens had to be fabricated and were made in accordance with the specifications of ASTM D 4123-82. The indirect tensile strength results for identical 4 inch and 6 inch samples should theoretically be the same. However, due to assumptions of homogeneity, isotropy, elasticity, etc., it is doubtful that the results from the two samples would be equal. The indirect tensile test results are shown in Tables 9 and 10.

Resilient Modulus Test

The resilient modulus tests were conducted on three specimens for each gradation using two load levels at three different temperatures for each load. The temperatures were 41 deg. F., 77 deg. F., and 104 deg. F. The two load levels were 10 percent and 15 percent of the indirect tensile strength at 77 degrees F. The procedure used for this test was ASTM D 4123-82 and the value for Poisson's Ratio used in calculating the test results was assumed to be 0.35. The load pulse duration was 0.10 sec. and the frequency was 1 pulse per second. The resilient modulus test results are shown in Tables 11, 12, 13, 14, and 15.

Table 9

Indirect Tensile Test Results for 4 Inch Diameter Specimens

Max. Agg. Size (in.)	Asp. Con. (%)	Spec. Ht. (in.)	Indirect Tensile Str. (psi)
3/8	4.5	2.471	141.7
3/8	4.5	2.488	124.7
3/8	4.5	2.499	141.7
Avg.			136.0
1/2	5.0	2.507	134.9
1/2	5.0	2.496	140.3
1/2	5.0	2.493	140.4
Avg.			138.5
3/4	4.3	2.468	158.0
3/4	4.3	2.476	160.7
3/4	4.3	2.477	147.8
Avg.			155.5
1	3.8	2.462	137.4
1	3.8	2.471	140.1
1	3.8	2.470	128.9
Avg.			135.4
1 1/2	3.4	2.467	107.2
1 1/2	3.4	2.462	151.9
1 1/2	3.4	2.467	166.1
Avg.			141.7

Table 10

Indirect Tensile Test Results for 6 Inch Diameter Specimens

Max. Agg. Size (in.)	Asp. Con. (%)	Spec. Ht. (in.)	Indirect Tensile Str. (psi)
3/8	4.5	3.702	117.5
3/8	4.5	3.674	122.0
3/8	4.5	3.718	124.8
Avg.			121.5
1/2	5.0	3.714	108.6
1/2	5.0	3.720	111.9
1/2	5.0	3.709	113.0
Avg.			111.2
3/4	4.3	3.723	106.2
3/4	4.3	3.720	109.1
3/4	4.3	3.699	110.4
Avg.			108.6
1	3.8	3.697	120.5
1	3.8	3.665	118.7
1	3.8	3.718	104.7
Avg.			114.7
1 1/2	3.4	3.697	122.7
1 1/2	3.4	3.710	123.7
1 1/2	3.4	3.707	119.5
Avg.			121.9

Table 11

Resilient Modulus Test Results for 4 Inch Diameter Specimens Using 3/8 Inch Maximum Aggregate Size

Test No.	% of St.	Ht. (in.)	Resilient Modulus (ksi) Temperature (deg. F.) 41	77	104
1	10	2.475	2124	1214	97
2	10	2.476	2427	1416	101
3	10	2.494	2824	1059	106
Avg.			2458	1230	101
1	15	2.475	1588	1271	122
2	15	2.476	2123	1158	68
3	15	2.494	2121	1157	82
Avg.			1944	1195	91

Table 12

Resilient Modulus Test Results for 4 Inch Diameter Specimens Using 1/2 Inch Maximum Aggregate Size

Test No.	% of St.	Ht. (in.)	Resilient Modulus (ksi) Temperature (deg. F.) 41	77	104
1	10	2.503	1714	470	50
2	10	2.496	2246	431	41
3	10	2.503	1895	491	39
Avg.			1952	464	43
1	15	2.503	1687	352	36
2	15	2.496	1687	324	52
3	15	2.503	1929	415	30
Avg.			1768	364	39

Table 13

Resilient Modulus Test Results for 4 Inch Diameter Specimens Using 3/4 Inch Maximum Aggregate Size

Test No.	% of St.	Ht. (in.)	Resilient Modulus (ksi) Temperature (deg. F.) 41	77	104
1	10	2.485	2004	231	91
2	10	2.467	2027	221	54
3	10	2.479	2017	205	38
Avg.			2016	219	61
1	15	2.485	1806	335	65
2	15	2.467	1303	358	61
3	15	2.479	1210	343	69
Avg.			1440	345	65

Table 14

Resilient Modulus Test Results for 4 Inch Diameter Specimens Using 1 Inch Maximum Aggregate Size

Test No.	% of St.	Ht. (in.)	Resilient Modulus (ksi) Temperature (deg. F.) 41	77	104
1	10	2.462	2074	529	52
2	10	2.481	1850	586	40
3	10	2.464	1957	480	43
Avg.			1960	532	45
1	15	2.462	1883	416	29
2	15	2.481	1886	440	53
3	15	2.464	1601	417	34
Avg.			1790	424	39

Table 15

Resilient Modulus Test Results for 4 Inch Diameter Specimens
Using 1 1/2 Inch Maximum Aggregate Size

Test No.	% of St.	Ht. (in.)	Resilient Modulus (ksi) Temperature (deg. F.) 41	77	104
1	10	2.454	2604	1006	123
2	10	2.448	2208	762	88
3	10	2.437	2454	581	79
Avg.			2422	783	97
1	15	2.454	2368	771	90
2	15	2.448	2548	625	68
3	15	2.437	2370	614	95
Avg.			2429	670	84

Creep Test

The creep test was conducted by applying a static load to each specimen for one hour followed by unloading for one hour (3). Stresses for the four inch and 6 inch diameter specimens was 51.7 psi and 55.2 psi, respectively. The stress in the four inch specimens was selected to be as high as possible without resulting in failure. The stress in the six inch specimens was the result of adapting the load on the testing device to achieve a stress in the six inch specimens which was approximately equal to the stress in the four inch specimens. All creep tests were conducted at temperatures ranging from 75 to 78 degrees F. The creep test device is shown in Figure 6 and a typical creep test result curve is shown in Figure 7.

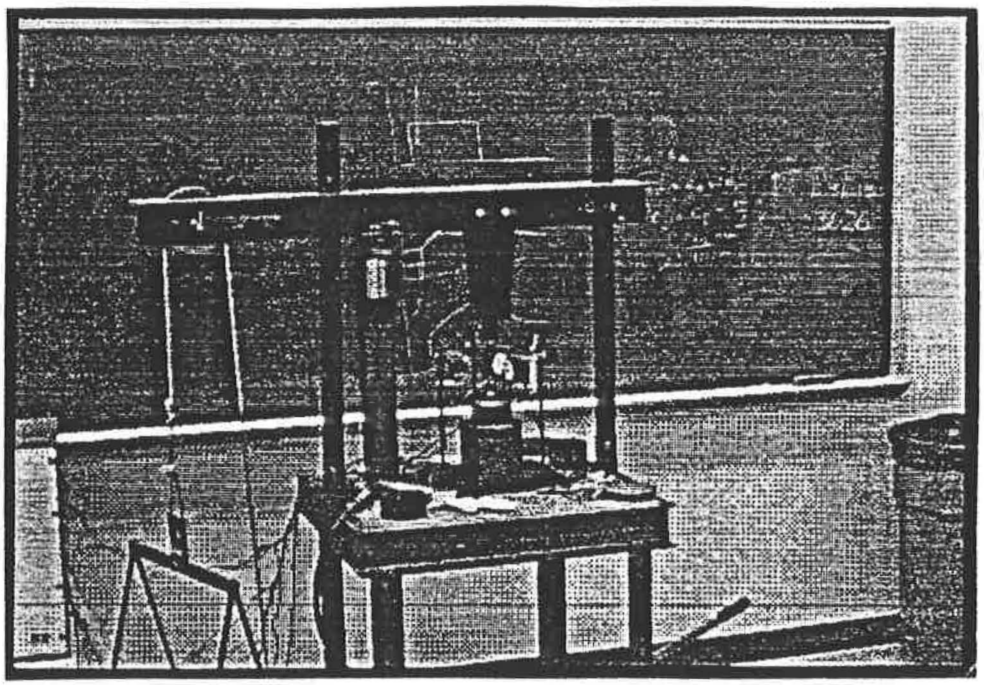

Figure 6. Creep Test Device

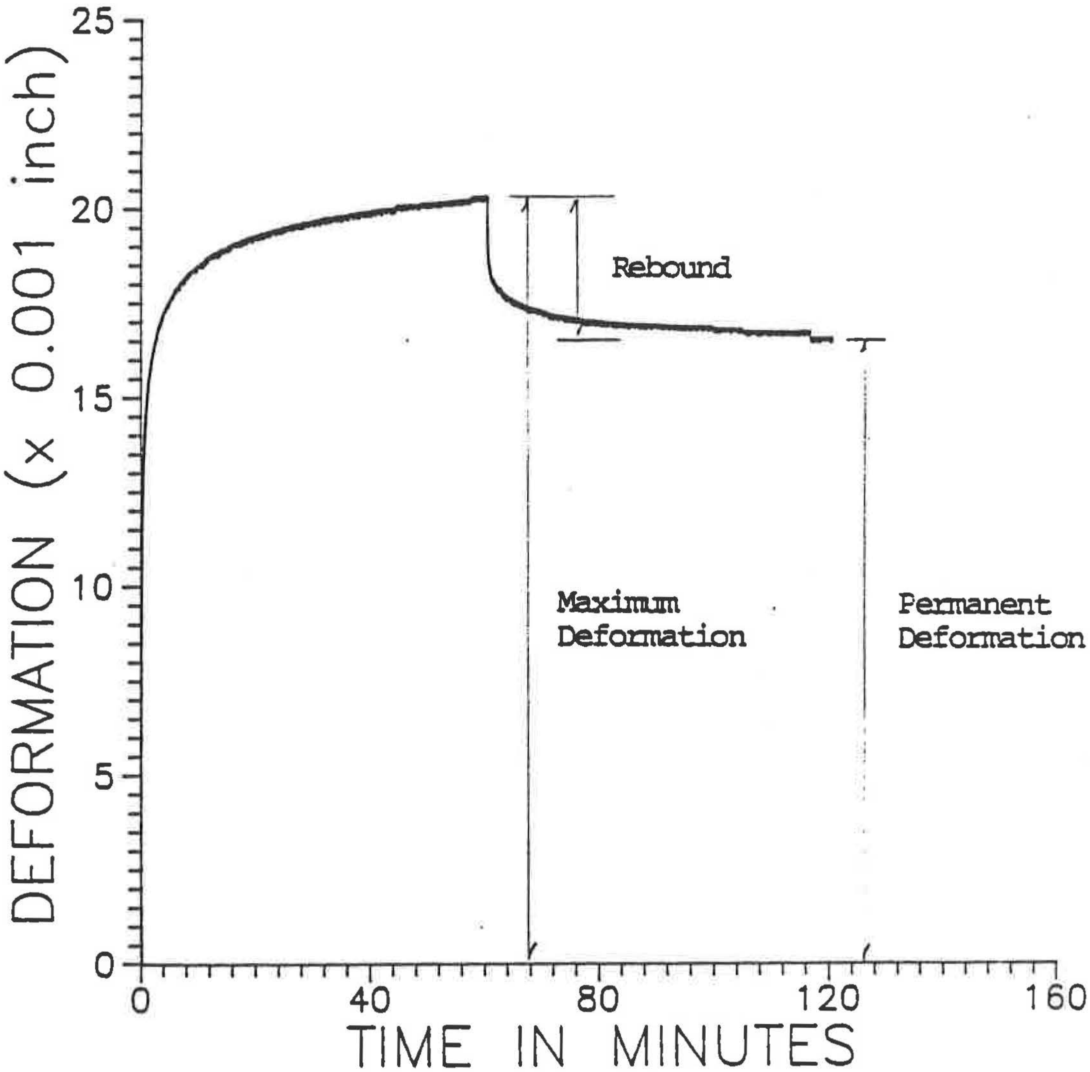

Figure 7. Typical Creep Test Result

These deformations were recorded at 1 second intervals using three linearly variable differential transducers (LVDT's).

Deformations for the creep tests were measured using LVDT's and recorded on data acquisition equipment. There was a loading plate mounted on top of each specimen in the test and the LVDT's were mounted against the loading plate at points equally spaced at intervals equal to 1/3 the circumference of the plate. The deformation at any time was determined by averaging the deformations of the three LVDT's used to make individual measurements. Creep test results are shown in Tables 16 and 17.

Table 16

Creep Test Results for 4 Inch Diameter Specimens

Max. Agg. Size (in.)	Spec. Grav.	Ht. (in.)	Max. Defor. (in.)	Rebound (in.)	Perm. Defor. (in.)
3/8	2.493	2.476	0.0139	0.0025	0.0115
3/8	2.490	2.479	0.0127	0.0029	0.0098
3/8	2.494	2.486	0.0105	0.0024	0.0080
Avg.			0.0124	0.0026	0.0098
1/2	2.503	2.518	0.0146	0.0024	0.0122
1/2	2.502	2.504	0.0128	0.0025	0.0102
1/2	2.514	2.505	0.0141	0.0025	0.0116
Avg.			0.0138	0.0025	0.0114
3/4	2.534	2.488	0.0215	0.0023	0.0192
3/4	2.481	2.525	0.0113	0.0017	0.0096
3/4	2.512	2.468	0.0133	0.0021	0.0112
Avg.			0.0154	0.0020	0.0133
1	2.521	2.472	0.0127	0.0020	0.0106
1	2.538	2.464	0.0131	0.0017	0.0114
1	2.533	2.485	0.0150	0.0024	0.0127
Avg.			0.0136	0.0020	0.0116
1 1/2	2.549	2.474	0.0087	0.0021	0.0065
1 1/2	2.530	2.476	0.0158	0.0016	0.0142
1 1/2	2.535	2.470	0.0293	0.0019	0.0275
Avg.			0.0179	0.0019	0.0161

Table 17

Creep Test Results for 6 Inch Diameter Specimens

Max. Agg. Size (in.)	Spec. Grav.	Ht. (in.)	Max. Defor. (in.)	Rebound (in.)	Perm. Defor. (in.)
3/8	2.480	3.763	0.0221	0.0038	0.0183
3/8	2.479	3.720	0.0198	0.0042	0.0156
3/8	2.473	3.751	0.0172	0.0034	0.0138
Avg.			0.0197	0.0038	0.0159
1/2	2.509	3.714	0.0247	0.0039	0.0208
1/2	2.503	3.729	0.0239	0.0046	0.0193
1/2	2.482	3.732	0.0211	0.0039	0.0171
Avg.			0.0232	0.0041	0.0191
3/4	2.511	3.699	0.0276	0.0045	0.0231
3/4	2.496	3.683	0.0261	0.0040	0.0221
3/4	2.519	3.689	0.0198	0.0037	0.0160
Avg.			0.0245	0.0041	0.0204
1	2.536	3.688	0.0195	0.0039	0.0156
1	2.545	3.686	0.0188	0.0032	0.0156
1	2.540	3.678	0.0203	0.0040	0.0163
Avg.			0.0195	0.0037	0.0158
1 1/2	2.564	3.699	0.0181	0.0035	0.0146
1 1/2	2.554	3.700	0.0180	0.0039	0.0141
1 1/2	2.559	3.663	0.0173	0.0038	0.0135
Avg.			0.0178	0.0037	0.0141

IV. ANALYSIS AND DISCUSSION OF TEST RESULTS

After completing tests on the asphalt mixtures, the results were analyzed to determine the expected effects on performance. Since this study only consisted of a laboratory evaluation, actual performance of the various asphalt mixtures was not established.

The gradation for the 3/8 inch maximum size aggregate contained approximately 2-3 percent (8.2% compared to 5.2-6.1%) more minus #200 material than the other gradations. The 0.45 power curve originally calculated a minus #200 content higher than this but the amount was lowered to meet the FHWA specifications. The amount was still much higher than the other gradations even though it had been lowered. The high dust content appeared to affect the test results more than the change in maximum aggregate size and hence the mixes with 3/8 inch maximum aggregate size were eliminated from the analysis.

Marshall Stability and Flow Tests

The results of the Marshall stability test seem to show trends similar to results as Huber and Heiman (13) showed. They reported no connection between stability and rutting resistance and the results of the tests for this study indicated that there was a poor relationship between Marshall stability and the maximum size of the aggregate. The linear regression in Figure 8 is almost horizontal with a coefficient of determination of 0.42. Since the regression line is approximately horizontal, there is little significant difference in the stability value for the various aggregate sizes evaluated. It could be argued that the result of the mixes using 1 1/2 inch maximum size aggregate should be ignored because mixes using the aggregate were larger than that allowed by the specified procedure. Even if that is done, the remaining three points show about the same trend between Marshall stability and aggregate size.

The relationship between flow and aggregate size (Figure 9, R^2 = .95) appears to be better than that for stability. Since flow is vertical deformation of the specimen in hundredths of an inch, it appears that larger aggregate in an asphalt concrete mix produced more vertical deformation, which indicates increased flexibility with increased aggregate size. All of the measured flow values are between 12 and 15 which is normal for typical asphalt mixtures.

Indirect Tensile Test

The indirect tensile test was one of the tests in which both six inch and four inch diameter specimens were tested (Figure 10). The two specimen sizes in Figure 10 indicated that there was very little change in indirect tensile strength as the maximum aggregate size changed. Even though the 6 inch specimens had a high R^2 value of 0.83, the increase in strength was only approximately 10% as maximum aggregate size increased from 1/2 to 1 1/2 inches.

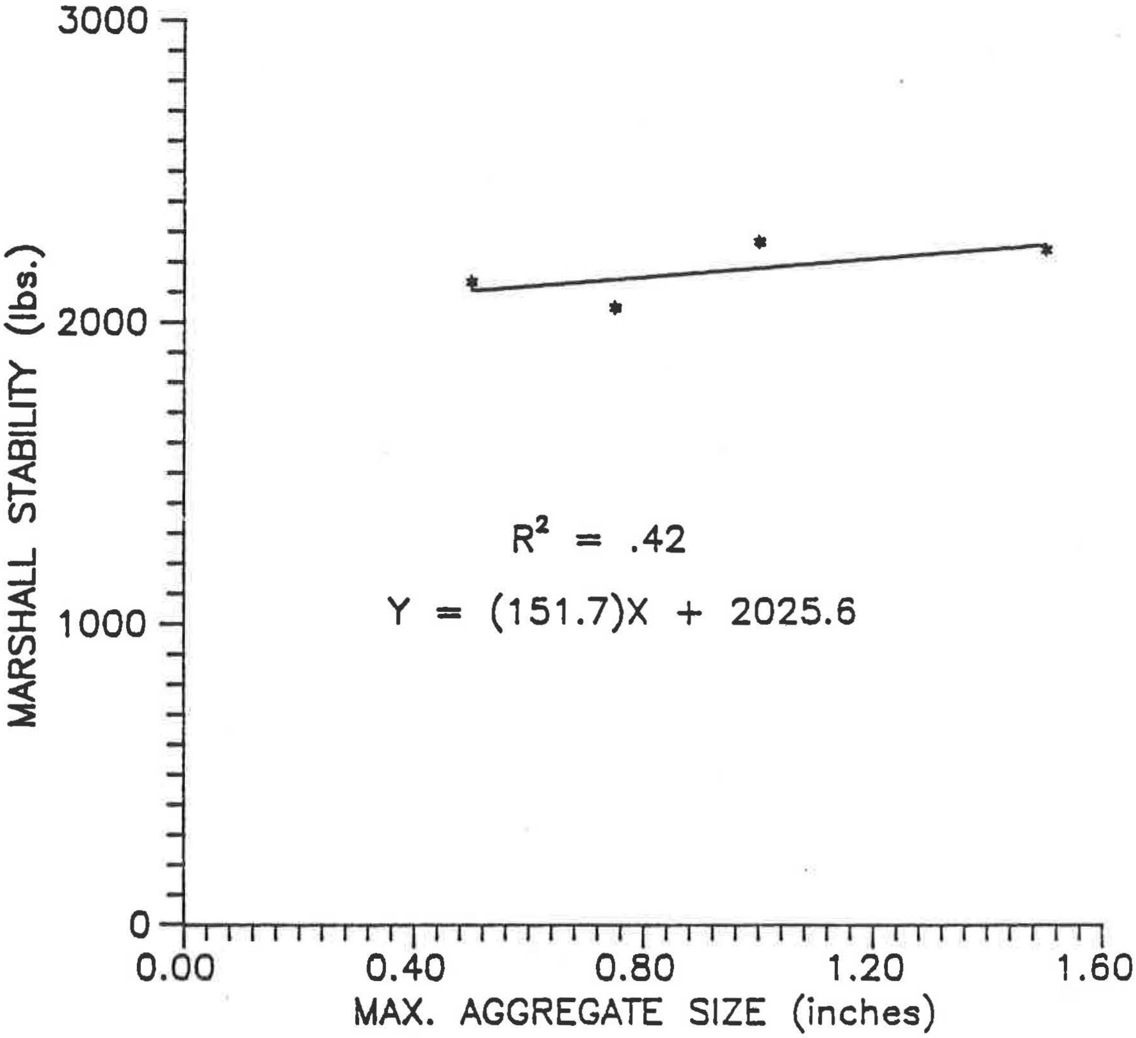

Figure 8. Marshall Stability for Four Inch Diameter Specimens

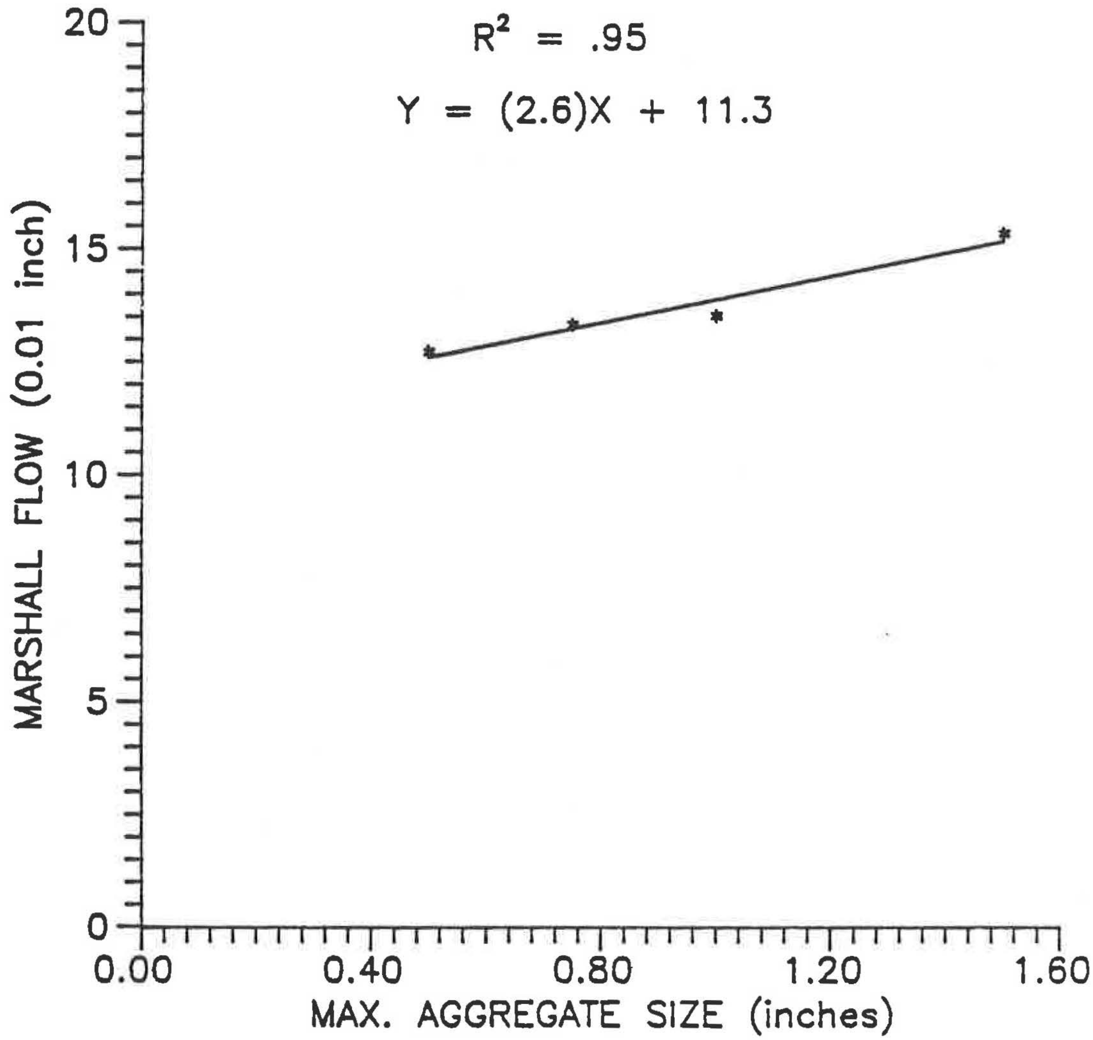

Figure 9. Marshall Flow for Four Inch Diameter Specimens

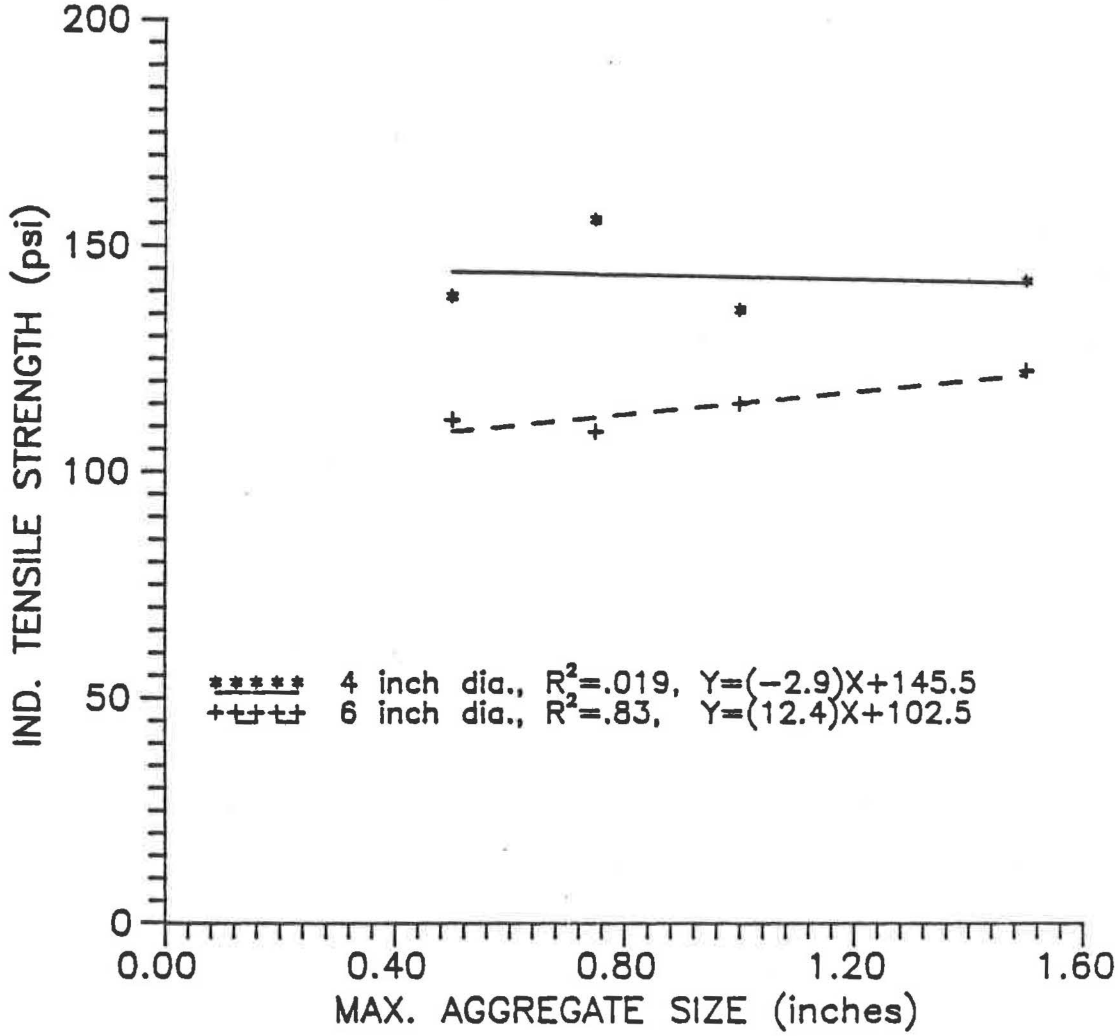

Figure 10. Indirect Tensile Test

Little change in tensile strength with changes in aggregate gradation was expected since tensile strength should be more affected by stiffness of the asphalt cement than by aggregate properties.

Figure 10 also shows that the tensile strengths for the 6 inch diameter specimens were always lower than the 4 inch diameter specimens. One of the differences between the two tests for the specific diameters was in strain rate. Since the loading rate (2 inches per minute) was the same for both sets of specimens, the strain rate for the 6 inch diameter specimens was 50% lower than that for the 4 inch diameter specimens. A lower loading rate should produce a lower tensile strength in the 6 inch diameter specimens and this was the case for every mix evaluated.

The 6 inch diameter also showed higher tensile strength for higher maximum aggregate size while the 4 inch diameter specimens showed opposite trends. Because of the higher R^2 value for the 6 inch diameter specimens, it appears that the data for 6 inch specimens is more precise.

The tensile strain at failure for the various mixes was analyzed for the gradations and the results are show in Figure 11. The tensile strain at failure, which is a measure of flexibility, was calculated from the vertical deformation at failure. There was approximately a 15 percent decrease in flexibility for the 6 inch diameter specimens (tensile strain at failure) going from the 1/2 inch maximum size gradation tensile test results to the 1 1/2 inch maximum size gradation tensile test results. The 4 inch diameter specimens showed approximately a 10% increase in flexibility but at a much lower R^2

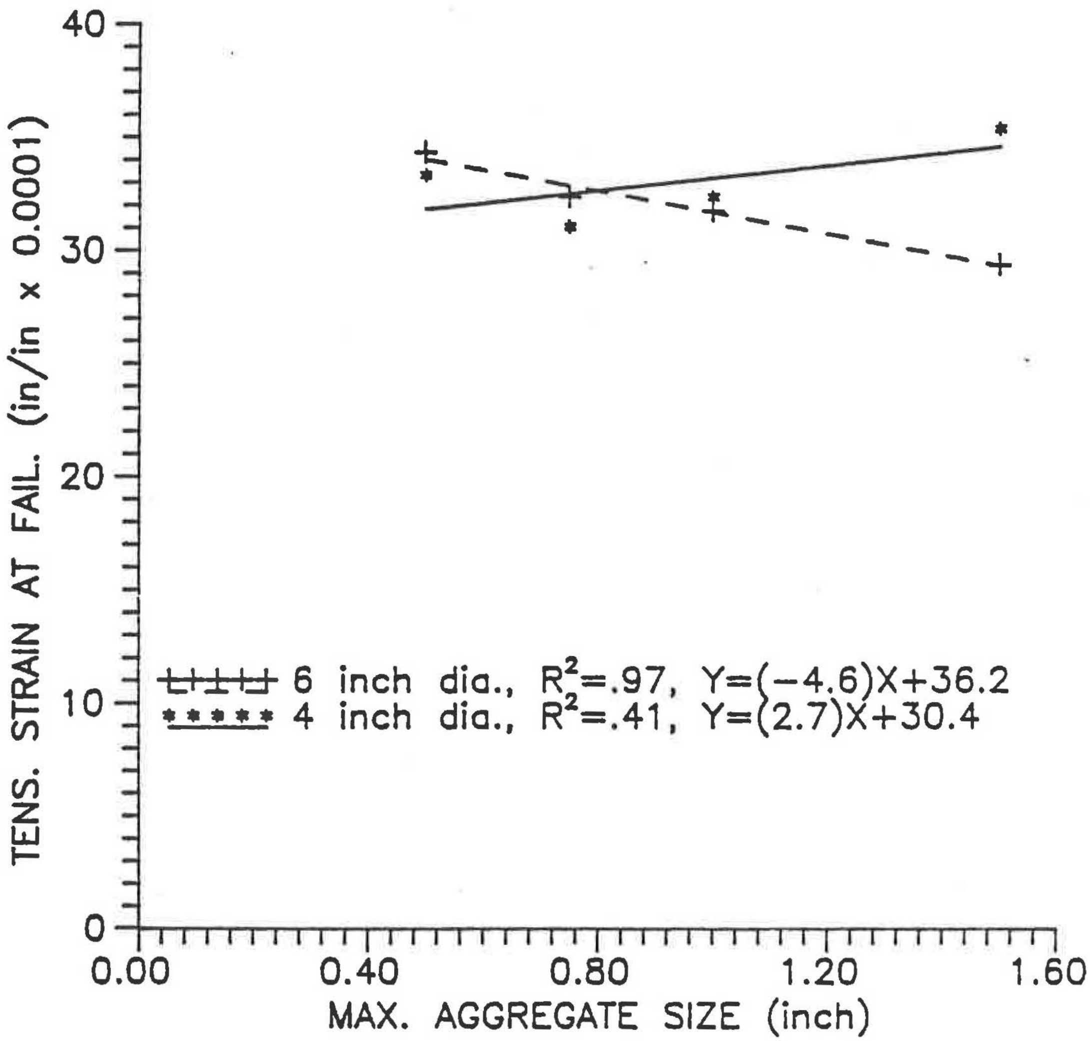

Figure 11. Tensile Strain at Failure

value than the 6 inch diameter specimens (0.41 versus 0.97). The 6 inch specimens showed a loss in flexibility for mixes with coarser aggregate which is opposite the results shown in the flow tests. Regardless, the loss or gain in flexibility for coarser mixes was insignificant and hence should not be considered an advantage or disadvantage.

Creep Test

The creep test data plotted in Figures 12, 13, and 14 indicates that the 4 inch and 6 inch diameter specimens give opposing results. Creep stiffness (Figure 14) was calculated by dividing the creep stress by the maximum strain at 60 minutes (21), permanent strain (Figure 12) was calculated by dividing the permanent deformation at 120 minutes by the original height of the test specimen, and percent rebound (Figure 13) was determined by dividing the total rebound by the maximum deformation at 60 minutes.

The 4 inch diameter samples in Figures 12, 13, and 14 show a decrease in strength with an increase in aggregate size and the 6 inch diameter samples show that strength increases with increased aggregate size. Results for the 4 inch diameter specimens are likely unduly influenced by the 1 1/2 inch maximum size mix. The variation in permanent strain for this mix for the 4 inch specimens is shown at the bottom of Table 18. For the 4 inch diameter specimens, the percent change in permanent strain for the 1 1/2 inch maximum aggregate size ranged from 70.90 percent above the average of all the tests conducted for that mix to 59.27 percent below the average.

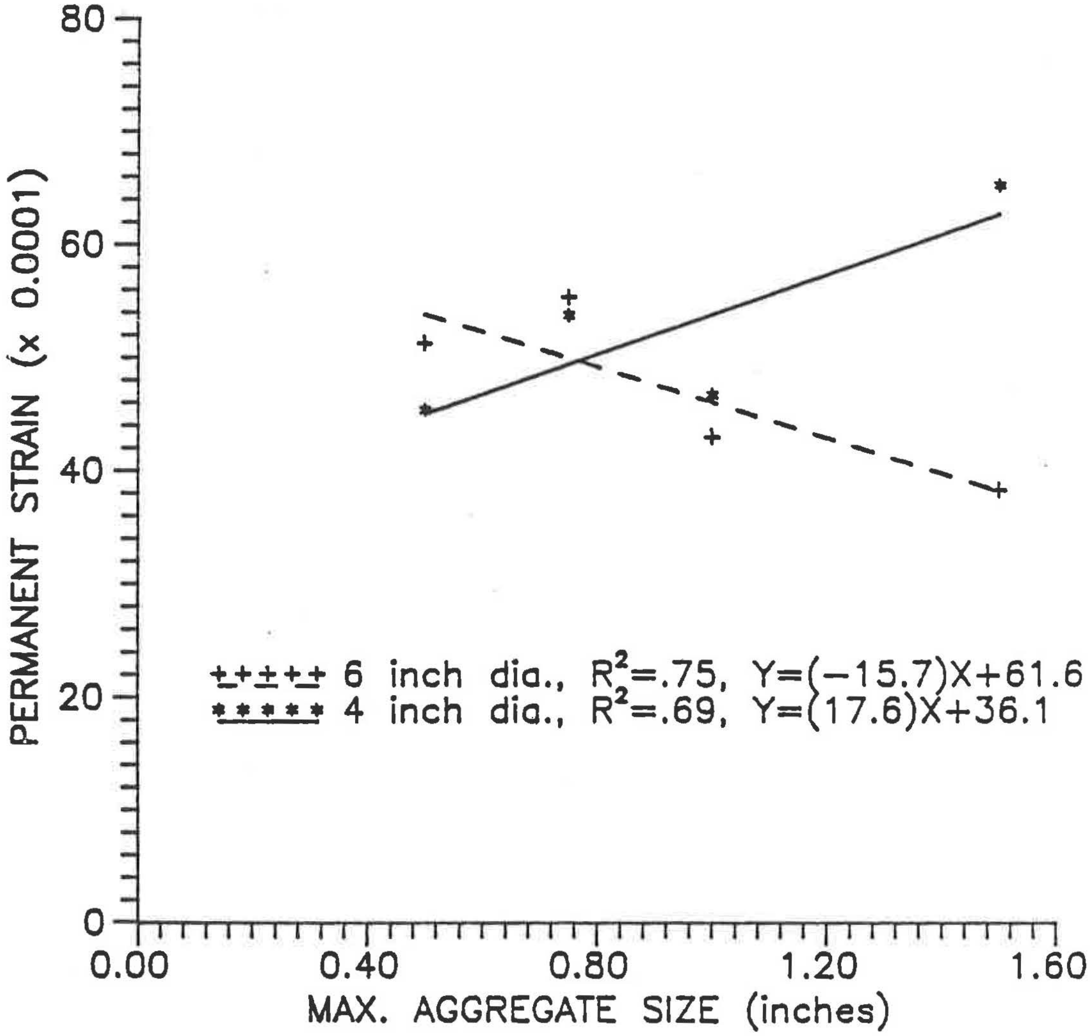

Figure 12. Average Permanent Strain for 4 Inch and 6 Inch Diameter Creep Tests

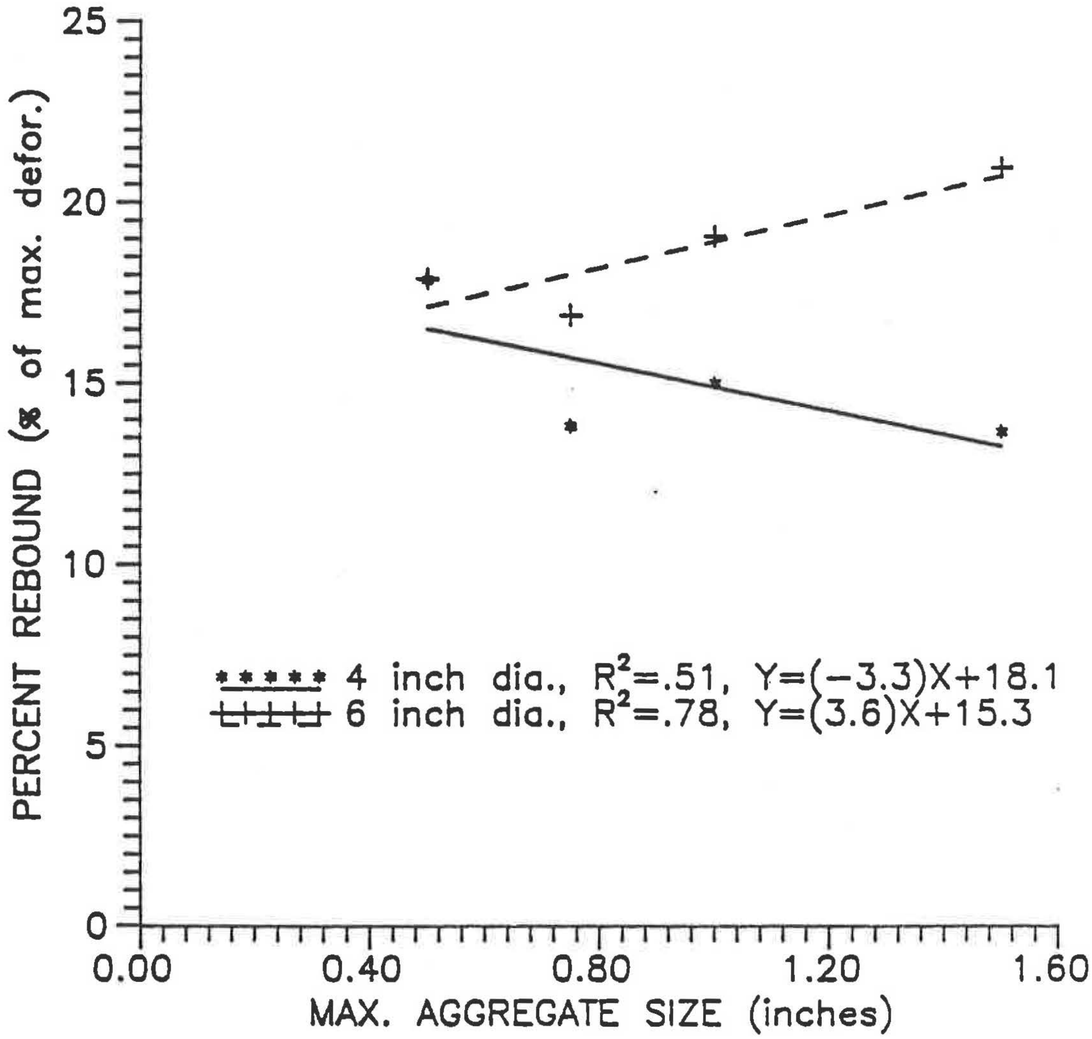

Figure 13. Average Percent Rebound for 4 Inch and 6 Inch Diameter Creep Tests

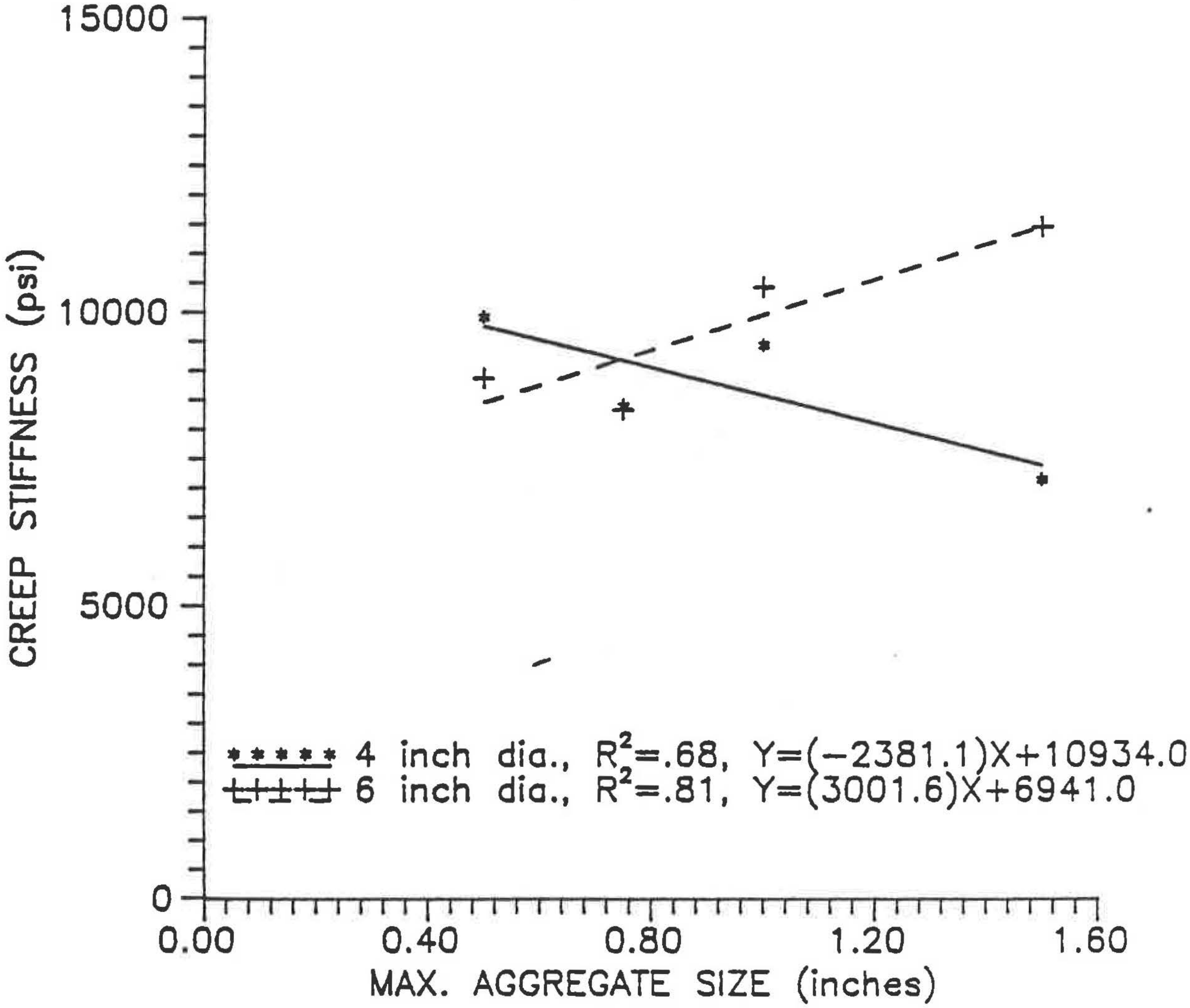

Figure 14. Average Stiffness for 4 Inch and 6 Inch Diameter Creep Tests

Table 18

Effects of Changing Specific Gravity on Individual Static Creep Test Results

Max. Agg Size	4 in. Diameter Perm. Strain	4 in. Diameter Spec. Grav.	6 in. Diameter Perm. Strain	6 in. Diameter Spec. Grav.
3/8	.00463	2.493	.00486	2.488
3/8	.00397	2.490	.00419	2.479
3/8	.00323	2.494	.00367	2.473
Avg.	.00394	2.492	.00424	2.480
1/2	.00486	2.503	.00559	2.509
1/2	.00408	2.502	.00518	2.503
1/2	.00464	2.514	.00458	2.482
Avg.	.00453	2.506	.00512	2.498
3/4	.00456	2.534	.00625	2.511
3/4	.00381	2.481	.00599	2.496
3/4	.00773	.2512	.00453	2.519
Avg.	.00537	2.509	.00553	2.509
1	.00507	2.521	.00422	2.536
1	.00462	2.538	.00422	2.545
1	.00430	2.533	.00444	2.540
Avg.	.00466	2.531	.00249	2.540
1 1/2	.00265	2.549	.00395	2.564
1 1/2	.00575	2.530	.00382	2.554
1 1/2	.01112	2.535	.00369	2.559
Avg.	.00651	2.538	.00382	2.559

The 6 inch diameter specimens which contained 1 1/2 inch maximum size aggregate had a range from only 3.4% above the average to 3.4% below the average for these specimens. The only other mix which had a large range of results was the one with 3/4 inch maximum size aggregate and 4 inch diameter which ranged from 44.04% above the average to 29.1% below the average. The 6 inch diameter specimen for this same size aggregate had a range from 13.02% to -21.34%.

It must be concluded that a range in results this wide for the 1 1/2 inch maximum size mix for the 4 inch diameter specimens is too high. The 6 inch diameter specimens give a more accurate representation of the relationship among all the mixes.

Based on the results from the 6 inch diameter specimens, permanent strain decreased with increased aggregate size, and percent rebound and stiffness increased with increased aggregate size. Hence, increasing the aggregate size should result in an asphalt mixture that is more resistant to permanent deformation.

Resilient Modulus Test

The resilient modulus was measured for all mixes and evaluated for the effects of aggregate size. Figures 15 and 16 show resilient modulus for the various mixes plotted against test temperature for applied stress levels of 10 and 15% of indirect tensile strength, respectively. The mix with 1 1/2 inch maximum size aggregate maintained the highest resilient modulus and the mix with the 3/4 inch maximum size aggregate the lowest resilient

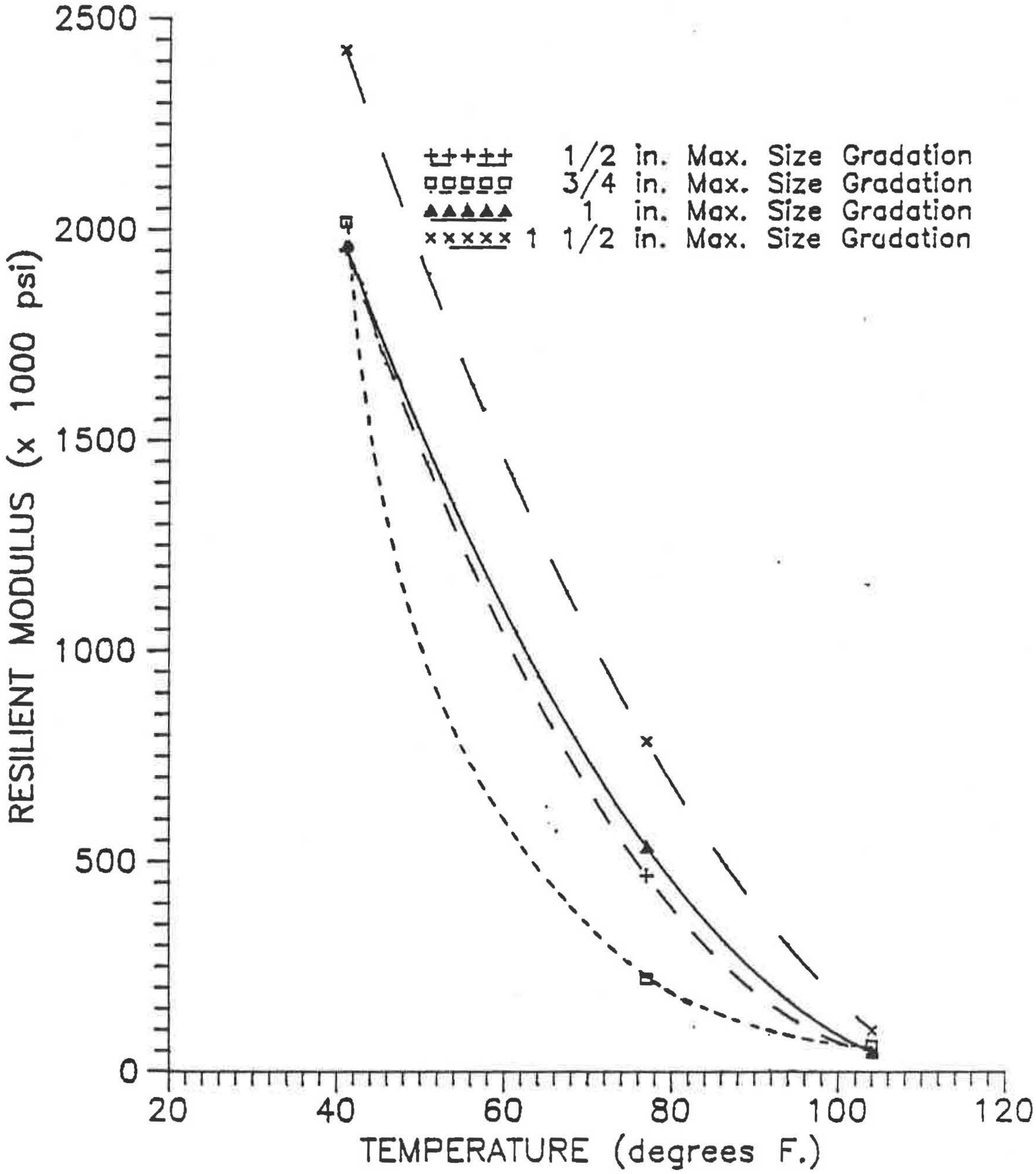

Figure 15. Effects of Temperature on Resilient Modulus at 10% of Indirect Tensile Strength Using 4 Inch Diameter Specimens

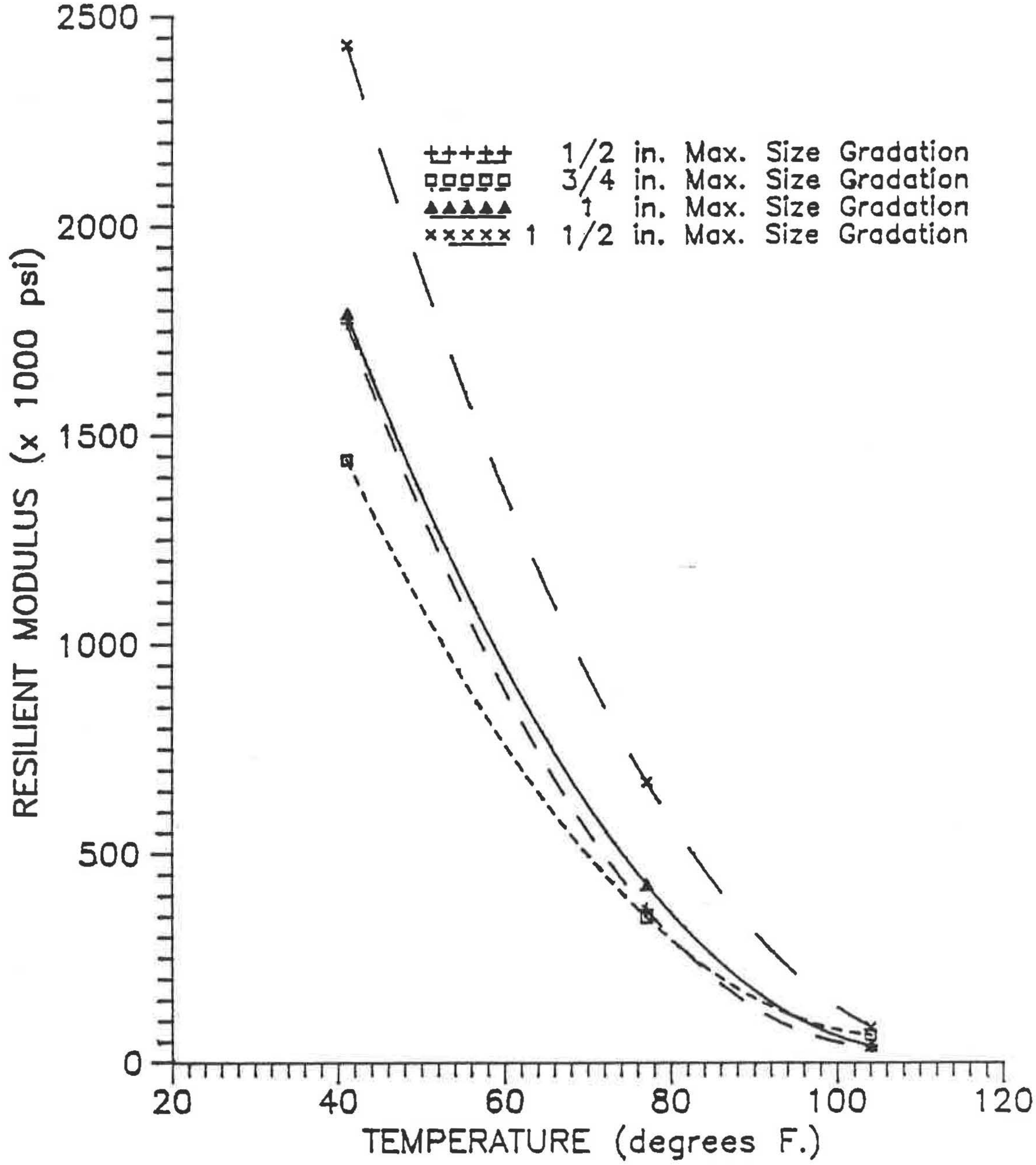

Figure 16. Effects of Temperature on Resilient Modulus at 15% of Indirect Tensile Strength Using 4 Inch Diameter Specimens

modulus at both stress levels. The mix with 1 inch maximum size aggregate was the second highest resilient modulus value for both stress levels and the mix with 1/2 inch maximum size aggregate was next to the lowest for both stress levels.

Figures 17 and 18 show that there is a good correlation between resilient modulus and maximum aggregate size (R^2 from .53 to .87). The resilient modulus increased as the maximum aggregate size increased from 1/2 to 1 1/2 inch. The resilient modulus at 10% of indirect tensile strength (Figure 18) increased about 25% at 41 deg. F., 133% at 77 deg. F. and 125% at 104 deg. F.

The stress level at 15% of indirect tensile strength (Figure 18) also yielded percentage increases in resilient modulus values when aggregate size increased from 1/2 to 1 1/2 inches. There was a 53% increase at 41 deg. F., 107% at 77 deg., and about 93% at 104 deg. F.

The increase in resilient modulus for larger maximum aggregate size will result in overall decreased pavement thickness required for given loading conditions. Hence, larger maximum aggregate size results in reduced overall pavement thickness.

Comparison of Six Inch and Four Inch Specimens

Comparison of the effects of specimen diameter on mix properties were performed using two tests--indirect tensile and creep. For 4 inch diameter specimens, the creep test (Figure 19) and the indirect tensile test (Figure 21) indicated much more variation in results for the 1 1/2 inch maximum aggregate size mixes than in results for mixes with 1 inch and smaller maximum

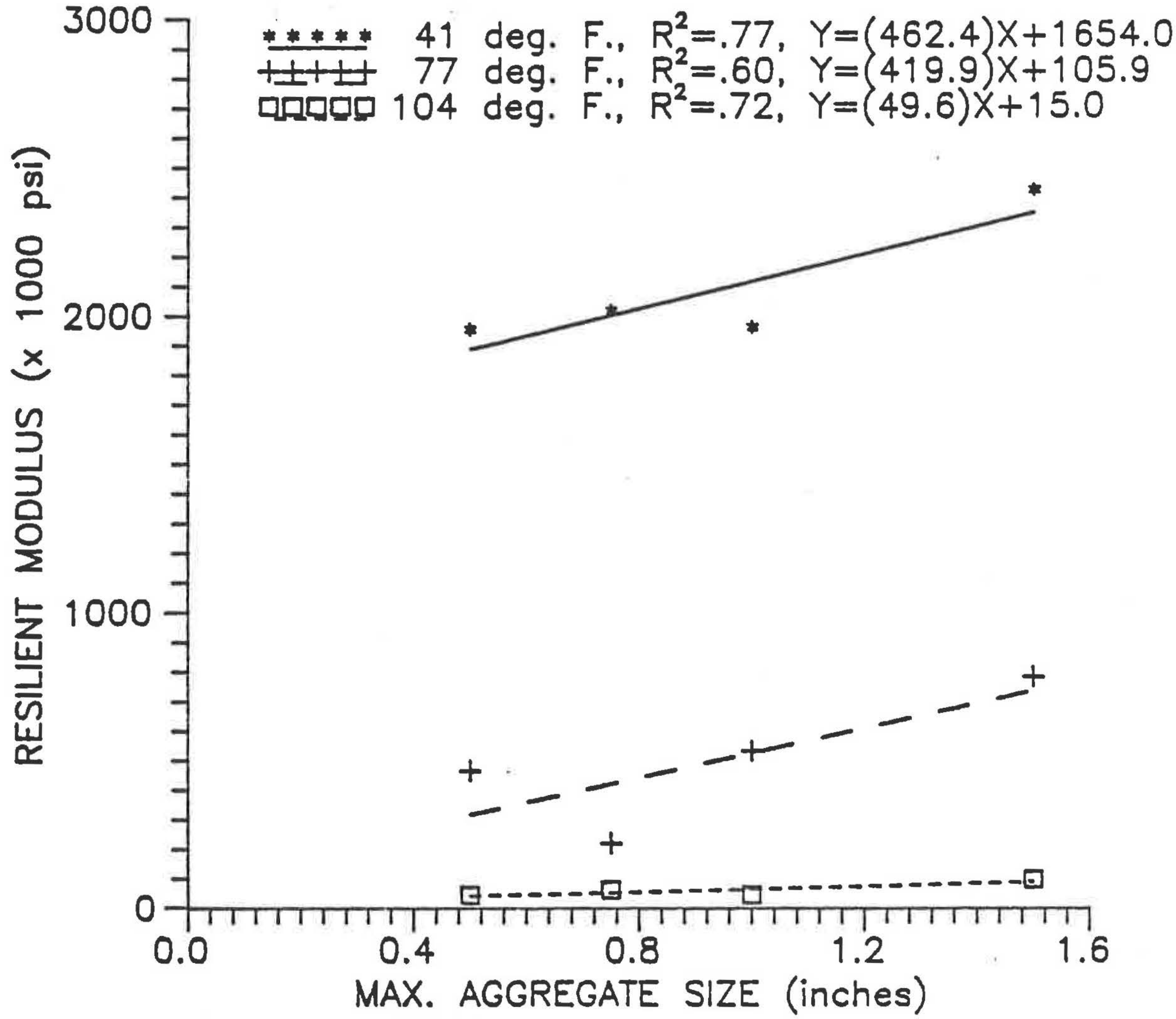

Figure 17. Change in Resilient Modulus with Respect to Maximum Aggregate Size for Different Temperatures at 10% of Indirect Tensile Strength

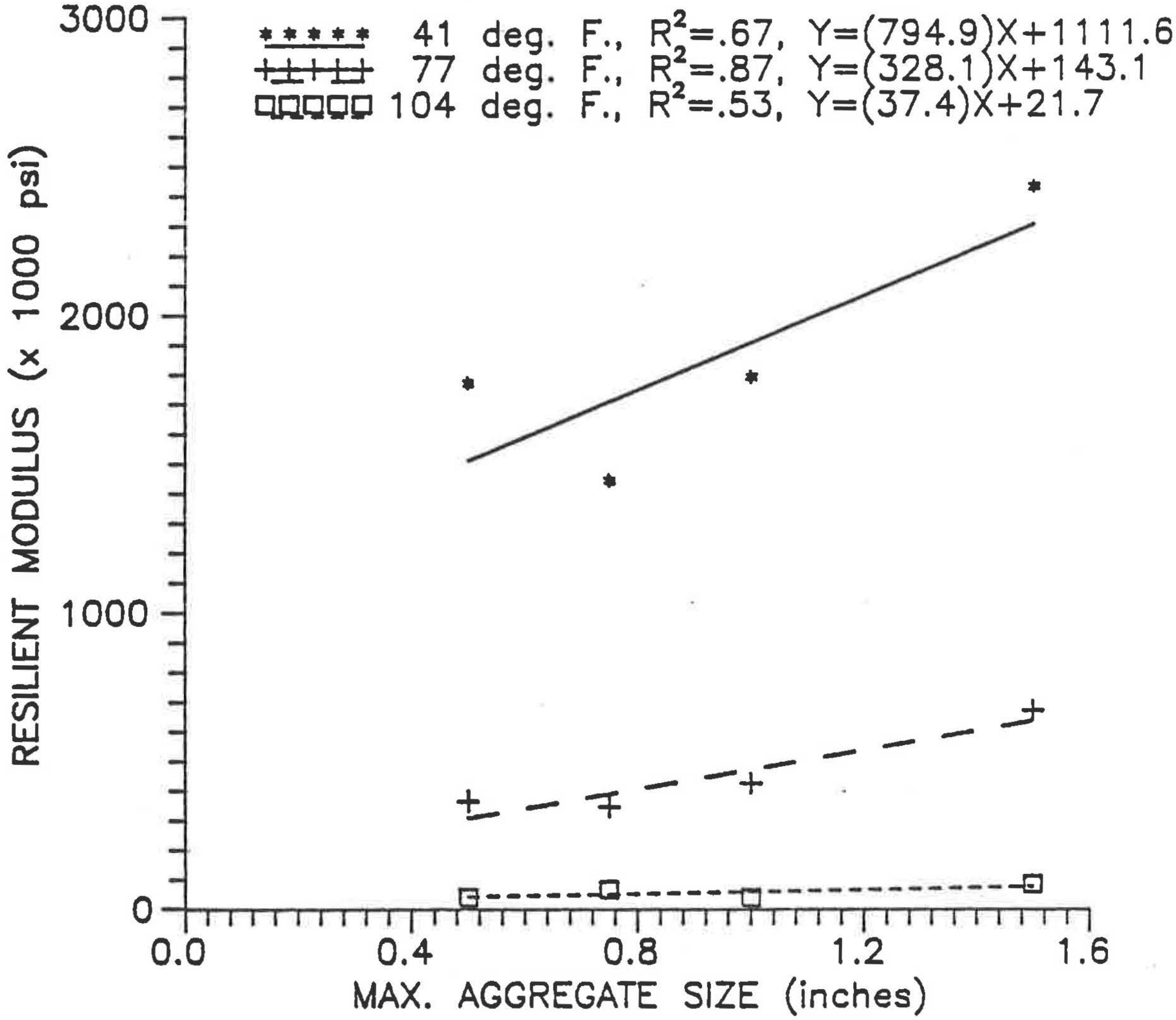

Figure 18. Change in Resilient Modulus with Respect to Maximum Aggregate Size for Different Temperatures at 15% of Indirect Tensile Strength

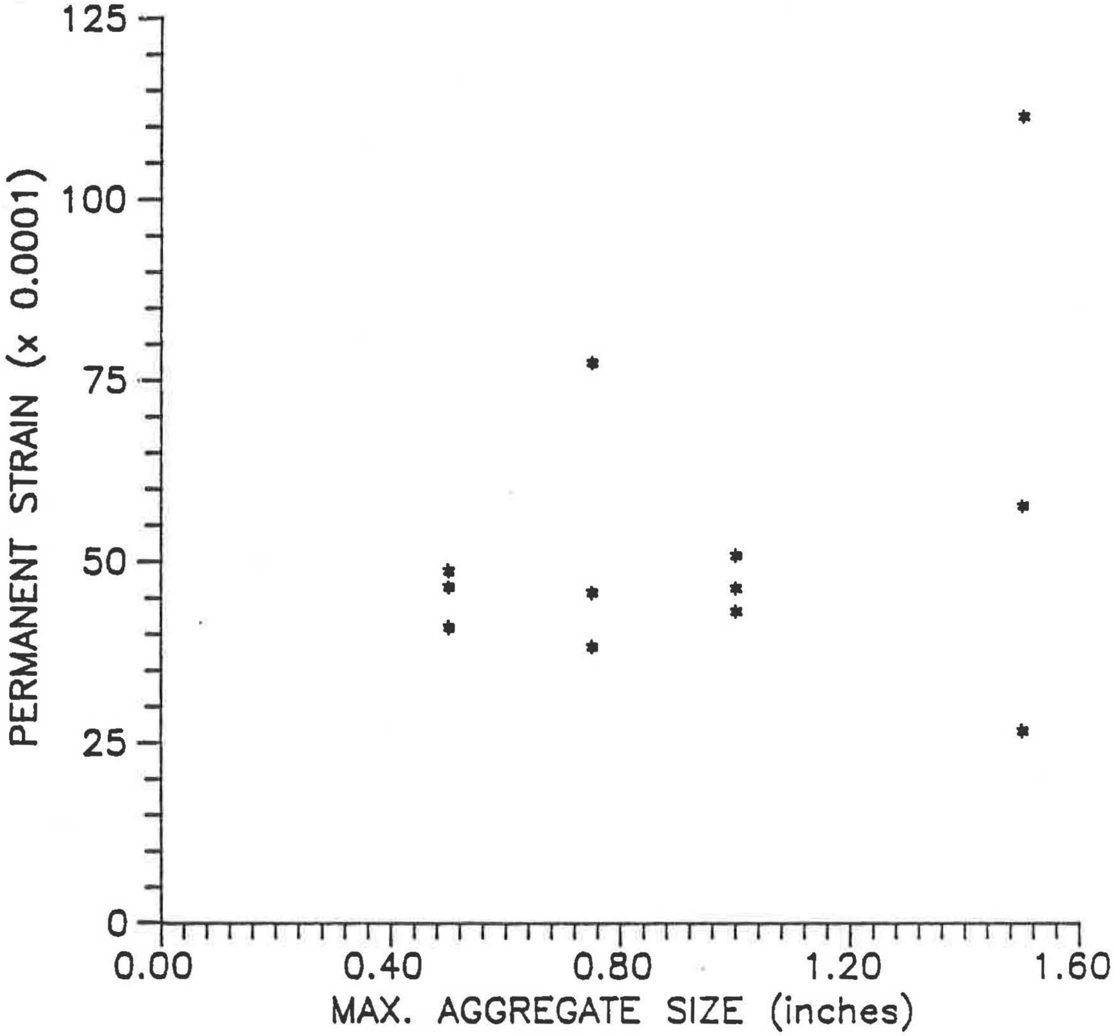

Figure 19. Individual Creep Test Results for 4 Inch Diameter Specimens

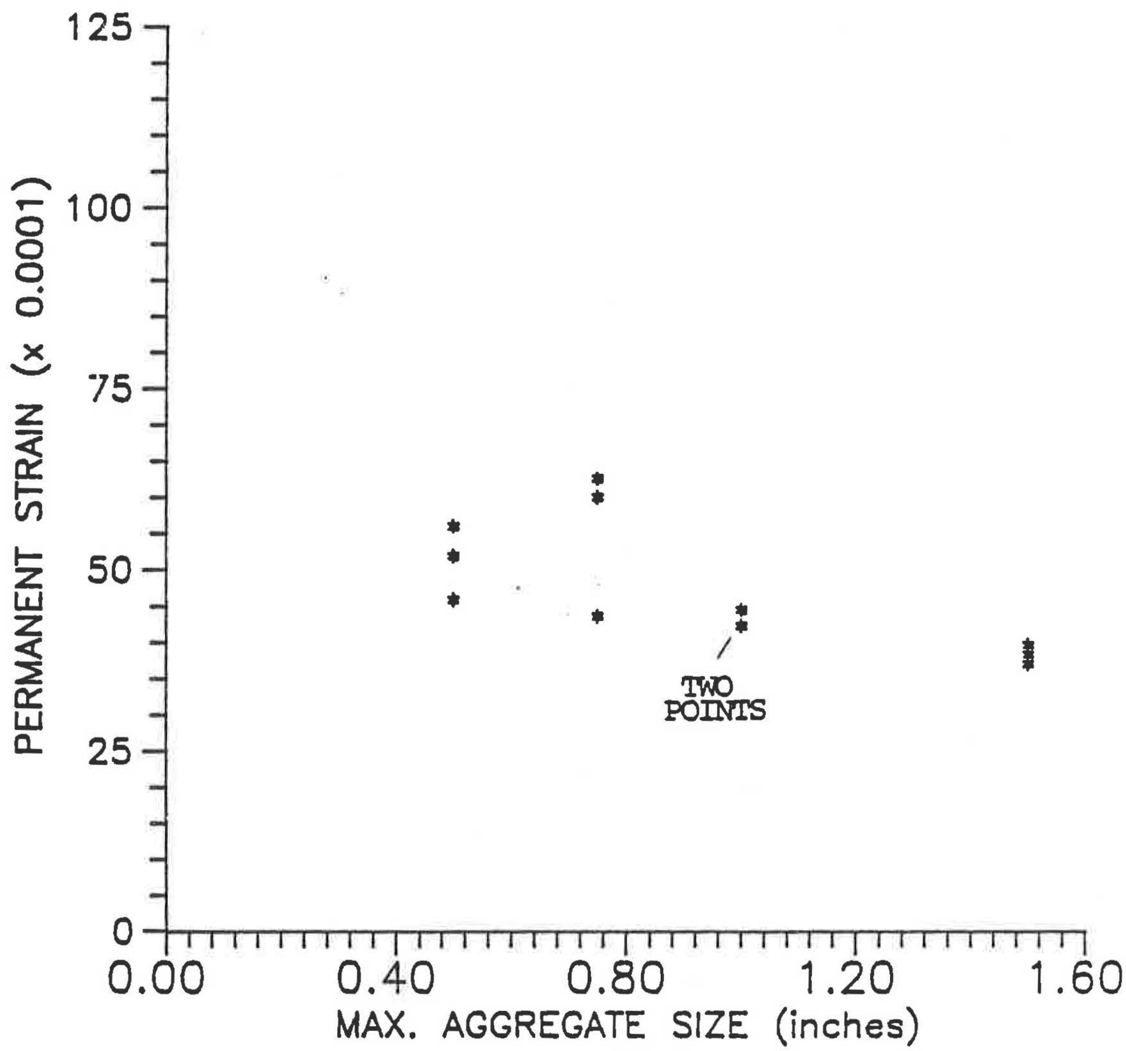

Figure 20. Individual Creep Test Results for 6 Inch Diameter Specimens

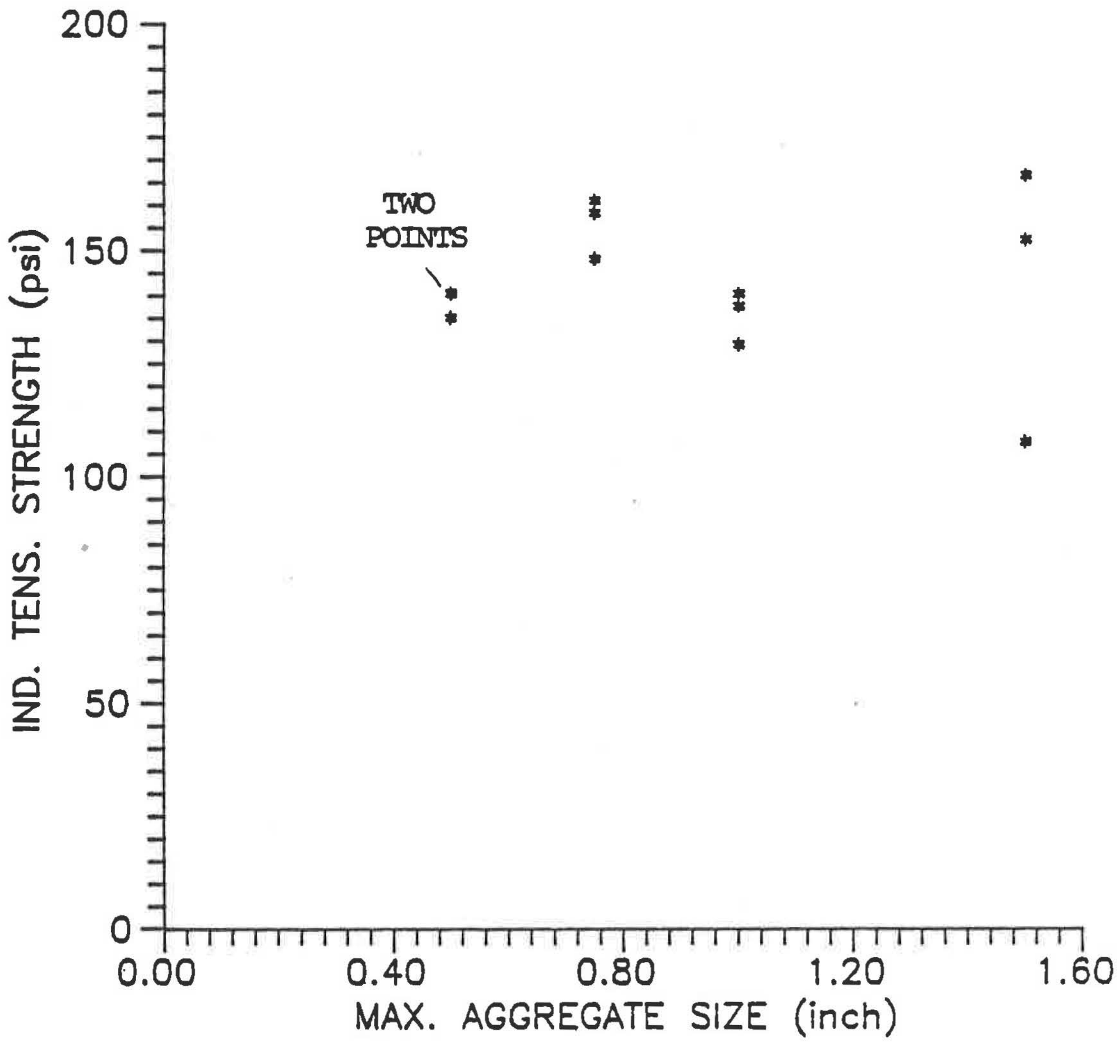

Figure 21. Individual Indirect Tensile Test Results for 4 Inch Diameter Specimens

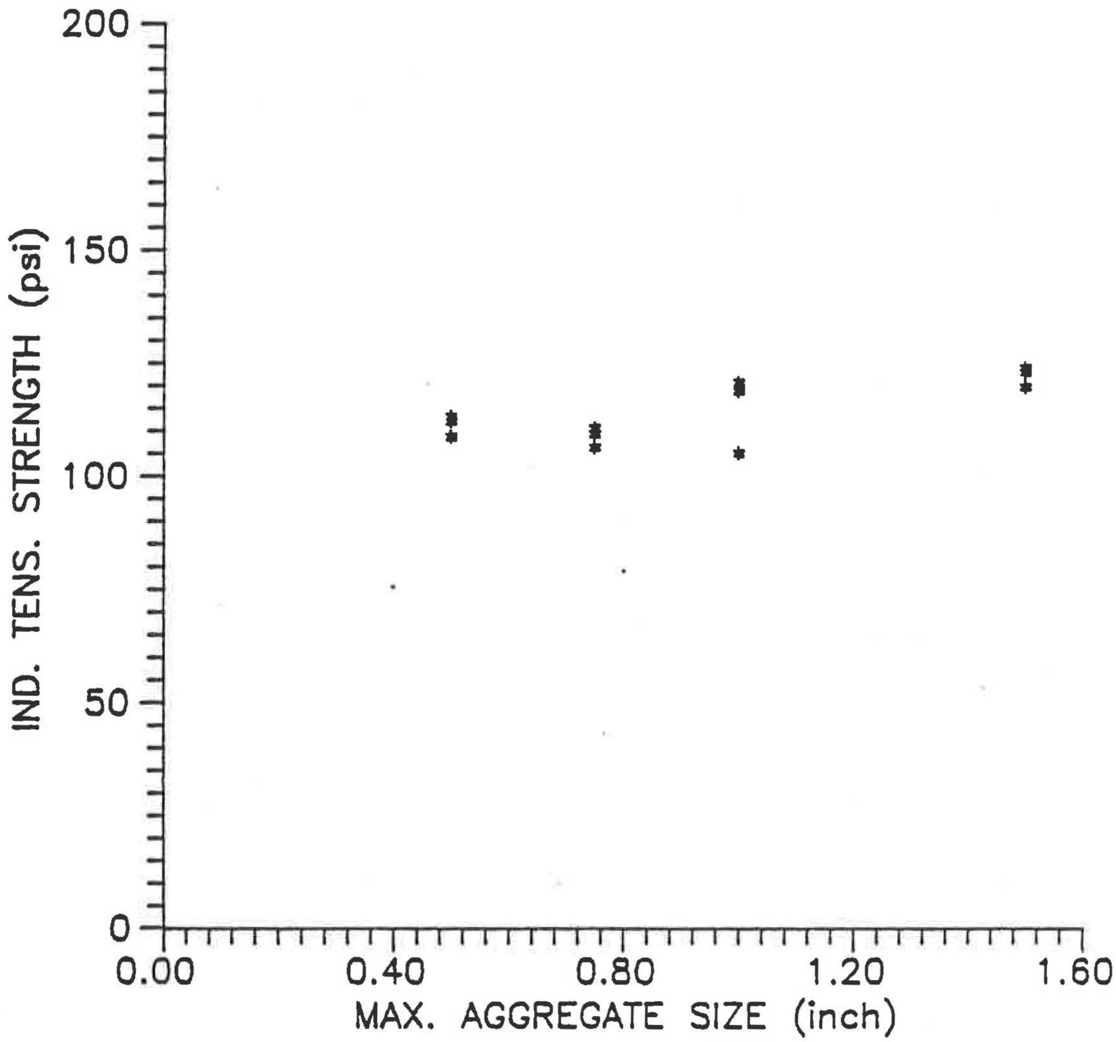

Figure 22. Individual Indirect Tensile Test Results for 6 Inch Diameter Specimens

aggregate size. The variability for 1 1/2 inch maximum aggregate size mixes was greatly reduced when 6 inch diameter specimens were used in testing (Figures 20 and 22).

This same reduction in variability using 6 inch diameter specimens rather than 4 inch or 1 1/2 inch maximum size aggregate was accomplished in tests by the Pennsylvania Department of Transportation and reported by Kandhal (15). In Kandhal's study, the coefficient of variation for Marshall stability as reduced from 11.1% for the 4 inch diameter specimens to 6.1% for the 6 inch. The coefficient of variation for flow was reduced from 21.6% to 6.8% when going from 4 inch diameter specimens to 6 inch diameter specimens.

The 6 inch diameter specimens also had lower variability for specimens using 3/4 inch maximum size aggregate for the creep test. The test results for the 3/4 inch maximum size mixes for the 4 inch diameter creep test (Figure 19) had approximately twice the range as that for the 6 inch diameter specimens (Figure 20).

Table 19 indicates that the 6 inch diameter specimens produced coefficients of determination that were consistently as high or higher than the 4 inch diameter specimens. This higher R^2 value for 6 inch diameter specimens indicates a better relationship between aggregate size and test properties and hence less error due to other causes such as random variability.

Table 19

Comparison of R2 Values for 6 Inch and 4 Inch Diameter Specimens

Description	6 Inch	4 Inch
Ind. Tensile Str. vs. Max. Aggregate Size	0.83	0.019
Tensile Strain at Failure vs. Max. Aggregate Size	0.97	0.41
Permanent Strain vs. Max. Aggregate Size	0.75	0.69
Percent Rebound vs. Max. Aggregate Size	0.78	0.51
Stiffness vs. Max. Aggregate Size	0.81	0.68
Specific Gravity vs. Max. Aggregate Size	0.93	0.93

Table 20 and Figure 23 indicate that the specific gravity values for the 4 inch and 6 inch diameter specimens are approximately equal for the 1/2 inch and the 3/4 inch maximum size aggregate but begin to diverge from one another for the other maximum aggregate sizes--especially for the 1 1/2 inch maximum size aggregate. This variation in density could have produced a divergence of results between the 4 inch and 6 inch diameter specimens for the creep and indirect tensile tests for the larger aggregate.

Effects of Gradation Changes on Costs

Table 21 and Figure 24 indicate how changes in the gradations used in this study affected the cost of the asphalt-aggregate mixtures. The cost figures were obtained from Engineering News Record and a quote from an aggregate supplier. The cost of the aggregate increased, as would be expected, when the specific gravity of the mixes increased for larger maximum size aggregate. However, the asphalt content required to maintain 4% air voids was reduced as the maximum aggregate size increased. Thus, the cost of asphalt-aggregate mix decreased accordingly with increased maximum aggregate size.

Table 21 shows how the increased dust content of the 3/8 inch maximums size gradation reduced asphalt content to such an extent that the total cost was lower than that for 1/2 inch maximum aggregate size mixes. This was contrary to the trend of the cost data for the other mixes.

Table 20

Specific Gravities of Six Inch and Four Inch
Diameter Specimens

Max. Agg. Size	4 inch dia. Spec. Grav.	6 inch dia. Spec. Grav.
3/8	2.492	2.477
1/2	2.505	2.501
3/4	2.514	2.508
1	2.531	2.540
1 1/2	2.540	2.557

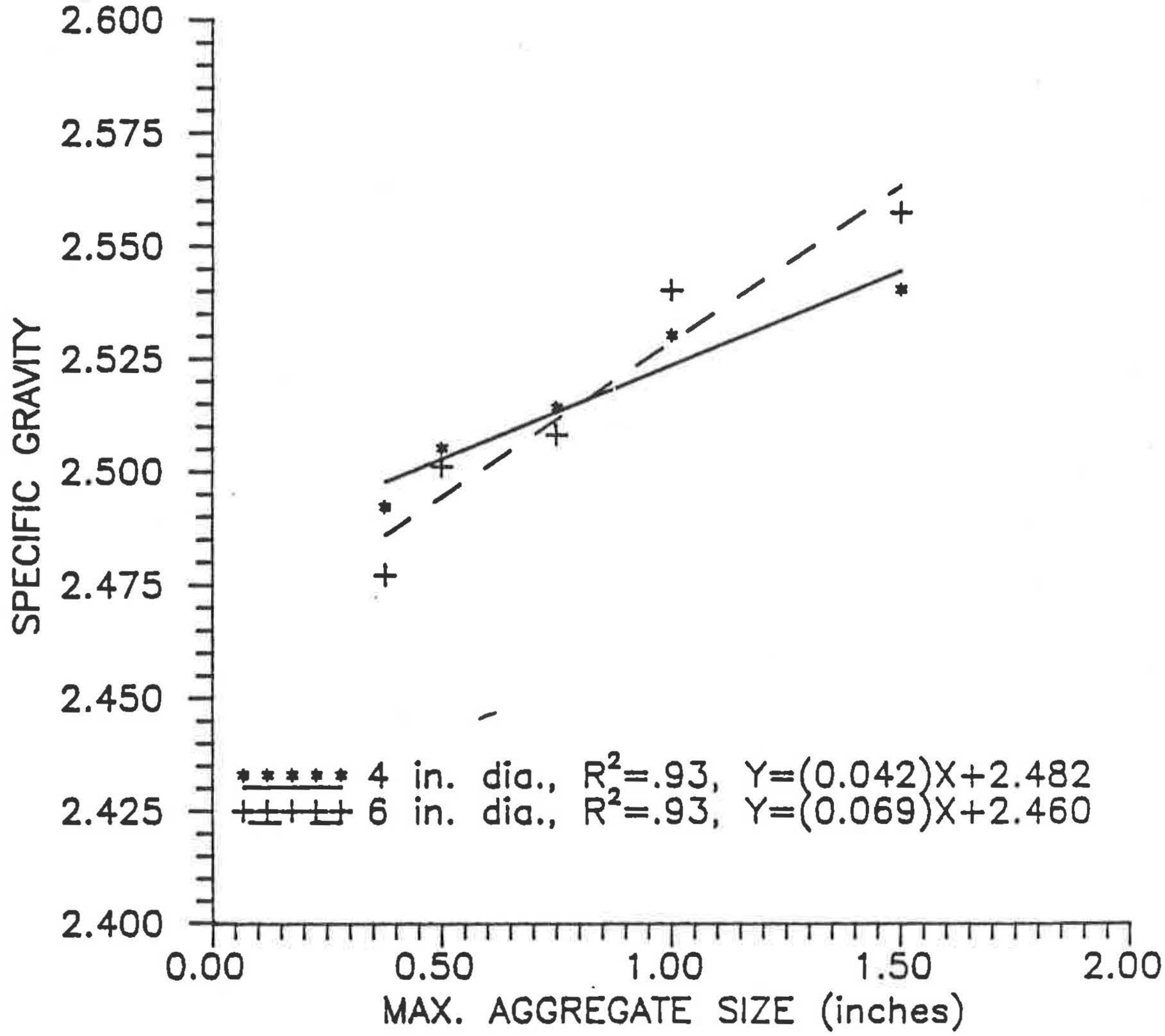

Figure 23. Average Specific Gravity for 4 Inch and 6 Inch Diameter Specimens Using Creep Test and Indirect Tensile Test Specimens

Table 21

Cost Analysis of Changes in Gradation

	Gradation Max. Size				
	3/8"	1/2"	3/4"	1"	1 1/2"
Asp. Con. at 4% voids (%)	4.52	4.95	4.25	3.82	3.37
Sp. Grav. at 4% voids	2.492	2.505	2.514	2.531	2.540
Asp./ton of mix (tons)	.0452	.0495	.0425	.0382	.0337
Agg./ton of mix (tons)	.9548	.9505	.9575	.9618	.9663
Asp. cost/ton of mix (1)	$4.803	5.260	4.516	4.059	3.581
Agg. cost/ton of mix (2)	$4.201	4.277	4.452	4.520	4.638
Total Materials cost/ton	$9.004	9.537	8.968	8.579	8.219
Materials cost/ sq. yd.*in.	$0.525	0.559	0.528	0.508	0.489

(1) ENR Magazine, 20 city average, (1989), (9) Asphalt cement, AC 20, $106.26 per ton.

(2) Prices f.o.b. Birmingham, Alabama area (total gradation costs of $4.40/ton for 3/8", $4.50 for 1/2", $4.65 for 3/4", $4.70 for 1", and $4.80 for 1 1/2")

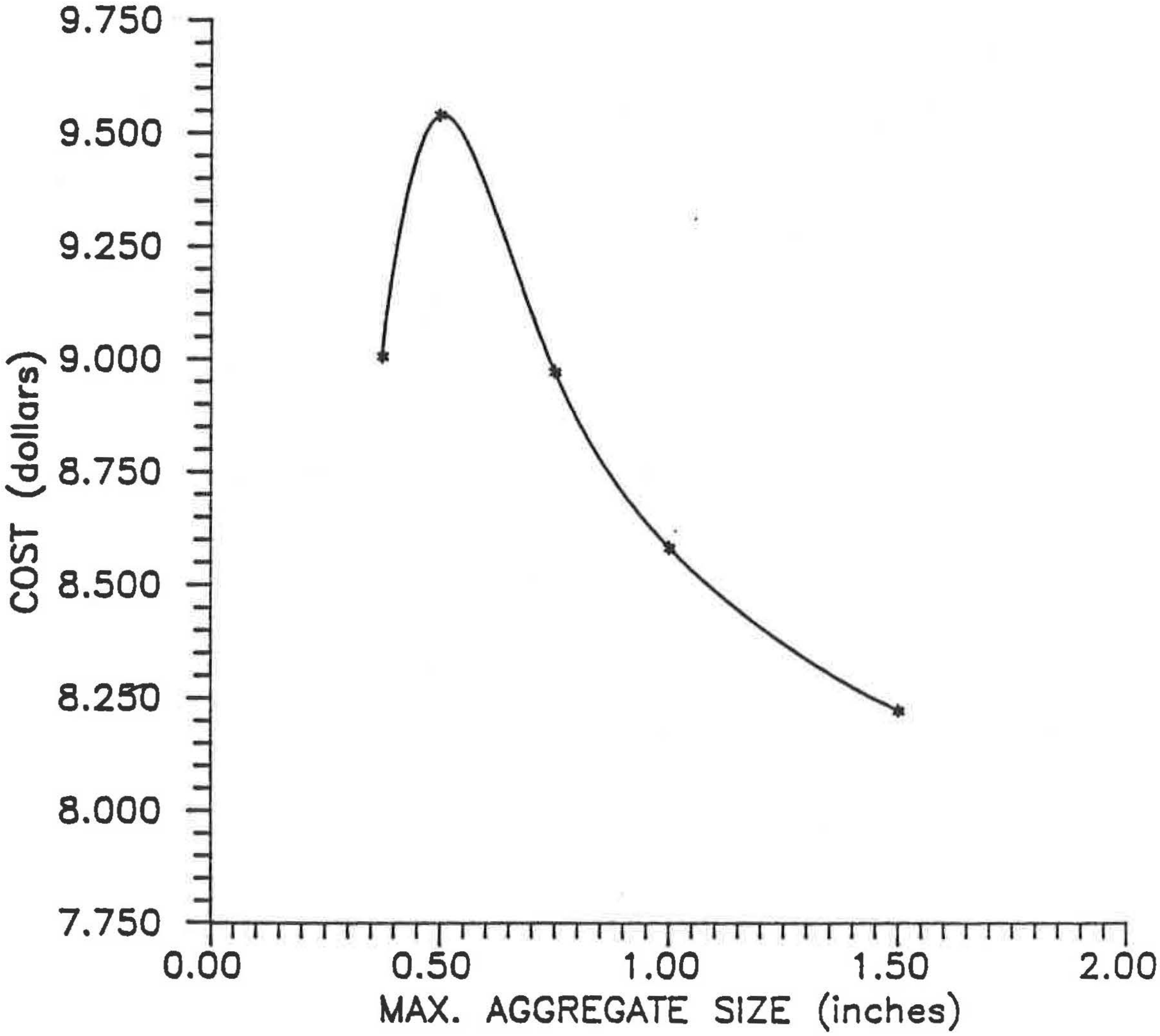

Figure 24. Asphalt-Aggregate Mix Cost Curve

Using mixes with 1 1/2 inch maximum size aggregate will result in savings of 10-20% in material costs when compared to mixes with smaller maximum size aggregate (3/8-3/4 inch maximum size mixes). This can result in substantial cost savings on larger projects.

V. CONCLUSIONS

The general trend of the data in this study shows that increasing the size of the largest aggregate in a gradation will increase the mix quality with respect to creep performance (6 inch specimens), resilient modulus (4 inch specimens), and tensile strength (6 inch specimens) but will not have a significant effect on Marshall stability. A high flow value was observed for mixes having larger maximum size aggregate.

Marshall stability from 4 inch specimens showed no significant relationship with maximum aggregate size. The stabilities were generally constant over the range of mixes evaluated in this study. The flow results, however, increased with the larger aggregate size which should result in more flexible mixes.

The indirect tensile test results showed a slight increase in tensile strength for increased maximum aggregate size. The strain at failure for the 6 inch specimens indicated decreased flexibility or ductility for larger maximum size aggregate but strain at failure for 4 inch specimens and Marshall flow indicated more flexibility for mixes with larger maximum size aggregate. Based on the combined results of flow and indirect tensile strength tests it appears that low temperature cracking would not be significantly affected by a change in aggregate size.

The static creep test using six inch diameter specimens showed greater rebound, more stiffness, and less permanent strain for larger maximum aggregate sizes. The four inch diameter specimens showed trends in the creep test results which were opposite the six inch diameter specimens. Based upon the 6 inch diameter creep test results, increased maximum aggregate size in a mix should increase the mix's resistance to rutting.

The resilient modulus increased with increased aggregate size. This means that increased maximum aggregate size will result in reduced strain in asphalt-aggregate mixtures when subjected to a given load in the field and therefore reduce stresses to the underlying layers.

The comparison for results for four inch and six inch diameter specimens indicated that results for six inch specimens were less variable than similar results for four inch diameter specimens. This may have been caused in part by the inadequacy of 4 inch diameter specimens for the mixes with larger maximum size aggregate, particularly the 1 1/2 inch maximum size mix. Steps need to be taken to standardize the use of 6 inch specimens and future work should utilize the 6 inch diameter specimens, especially when aggregate size greater than 1 inch is used. The 6 inch specimens generally showed improvement in mix properties for increased maximum aggregate size while the 4 inch specimens generally had an opposite trend (primarily as a result of the mixes with 1 1/2 inch maximum size aggregate).

This study showed that increasing the size of the largest aggregate in a mix reduces mix costs because of savings in the cost of asphalt cement. Material savings of 10-20% can be expected when using mixes with larger maximum size aggregate.

VI. RECOMMENDATIONS

Based on this study the following recommendations are made concerning any future work to evaluate mixes with varying gradations.

Tighter control on the minus #200 material should be exercised in future research relating to the effects of aggregate on the performance of a mix. The factor which led to the deletion of the 3/8 inch maximum size aggregate mixes from the analysis of the test results of this project was the inclusion of too much minus #200 material in the mix.

The effect of the loading rate (strain rate) on the results from the indirect tensile test for different diameter specimens should be evaluated. Changes in the strain rate resulting from a constant loading rate will likely produce different results (higher strain rates will produce higher tensile strength and vice versa).

Compaction in the field occurs over time and under normal conditions may reduce air voids in the asphalt-aggregate mix from 8-9% down to 4%. More work is needed in the laboratory to evaluate the properties of asphalt mixtures over a practical range of void contents that can be expected in the field.

The scope of this study was confined to all crushed materials. Particle shape can have an impact on the performance of an asphalt-aggregate mix and should be studied to determine its effects on the properties of asphalt mixtures.

REFERENCES

(1) Acott, Mike, Holt, Dave, and Puzinauskas, Vyt, (1988) "Design and Performance Study of a Heavy Duty Large Stone Hot Mix Asphalt Under Concentrated Punching Shear Conditions," published by the National Asphalt Pavement Association, Series 105/88.

(2) Anderson, David A., and Tarris, Joseph P., (1983) "Characterization and Specification of Baghouse Fines," Proceedings, Association of Asphalt Paving Technologists, Volume 52, pp. 88-120.

(3) Bolk, Ir. H. J. A., (1981) The Creep Test, Study Centre for Road Construction, The Netherlands, p. 9.

(4) Brown, E. R., (1987) "Mix Design and Construction of Asphalt Concrete to Support High Tire Pressures," paper prepared for presentation at AASHTO/FHWA Symposium on High Pressure Truck Tires, Austin, Texas, February 1987.

(5) Brown, E. R., McCrae, John L., and Crawley, Alfred B., (1986) "Effects of Aggregates on Performance of Bituminous Concrete," paper presented for A.S.T.M. Symposium on the Implication of Aggregate in the Design, Construction, and performance of Flexible Pavements, December 1986, New Orleans, Louisiana.

(6) Brown, S. F., and Cooper, K. E., (1984) "The Mechanical Properties of Bituminous Materials for Road Bases and Basecourses," Proceedings, Association of Asphalt Paving Technologists, Volume 53, pp. 415-439.

(7) Cross, Steven Alan, (1985) "The Development of a Rational Mix Design Method for Arkansas Black Bases," unpublished thesis for Master of Science in Civil Engineering degree at The University of Arkansas.

(8) Davis, Richard L., (1988) "Large Stone Mixes: A Historical Insight," published by the National Asphalt Pavement Association, Series 103/88, Riverdale, Maryland.

(9) (1989) "ENR Materials Prices," ENR Magazine, February 2, 1989, Volume 222, No. 5, p. 42.

(10) Evans, J. V., and Ott, L. E., (1986) "The Influence of Material Properties and Pavement Composition on Permanent Deformation by Flow," Proceedings, The Association of Asphalt Paving Technologists, Volume 55, pp. 353-373.

(11) Federal Highway Administration, (1985) "Standard Specifications for Construction of Roads and Bridges on Federal Highway Projects," U.S. Department of Transportation, FP-85, p. 595.

(12) Goode, Joseph F., and Lufsey, Lawrence A., (1962) "A New Graphical Chart for Evaluating Aggregate Gradation," Bureau of Public Roads, Physical Research Division, paper presented at the annual meeting of the Association of Asphalt Paving Technologists, New Orleans, Louisiana.

(13) Huber, G. A., and Heiman, G. H., (1987) "Effect of Asphalt Concrete Parameters on Rutting Performance: A Field Investigation," Proceedings, Association of Asphalt Paving Technologists, Volume 56, pp. 33-61.

(14) Kalcheff, I. V., and Tunnicliff, David G., (1982) "Effects of Crushed Stone Aggregate Size and Shape on Properties of Asphalt Concrete," Proceedings, The Association of Asphalt Paving Technologists, Volume 51, pp. 453-473.

(15) Kandhal, Prithvi S., (1989) Testing and Evaluation of Large Stone Mixes Using Marshall Mix Design Procedures, National Center for Asphalt Technology, Auburn University, draft of Interim Report.

(16) Kennedy, Thomas W., (1977) "Characterization of Asphalt Pavement Materials Using the Indirect Tensile Test," Proceedings, The Association of Asphalt Paving Technologists, Volume 46, pp. 132-150.

(17) Khalifa, Mohamed Osama, and Herrin, Moreland, (1970) "The Behavior of Asphaltic Concrete Constructed with Large-Sized Aggregate," Proceedings, Association of Asphalt Paving Technologists, Volume 39, pp. 345-376.

(18) Lai, James S., Hufferd, William L., (1976) "Predicting Permanent Deformation of Asphalt Concrete from Creep Tests," Transportation Research Record 616, Transportation Research Board, pp. 41-43.

(19) Marek, Charles R., and Herrin, Moreland, (1968) "Tensile Behavior and Failure Characteristics of Asphalt Cements in Thin Films," Proceedings, Association of Asphalt Paving Technologists, Volume 37, pp. 386-421.

(20) Regan, George L., (1987) "A Laboratory Study of Asphalt Concrete Mix Designs for High-Contact Pressure Aircraft Traffic," U.S. Army Engineer Waterways Experiment Station, Geotechnical Laboratory, Final Report.

(21) Van de Loo, P. J., (1974) "Creep Testing, a Simple Tool to Judge Asphalt Mix Stability," Proceedings, Association of Asphalt Paving Technologists, Volume 43, pp. 253-283.

(22) Waller, H. Fred, Jr., (1988) "Port on the Move," Asphalt Magazine, The Asphalt Institute, Volume 1, number 3, pp. 14-15.

(23) Wedding, Presley A., and Gaynor, Richard D., (1961) "The Effects of Using Crushed Gravel as the Coarse and Fine Aggregate in Dense Graded Bituminous Mixtures," Proceedings, Association of Asphalt Paving Technologists, Volume 30, pp. 469-487.

APPENDIX

TEST RESULTS

Table 22

Marshall Stability and Flow Results

TEST NO.	ASPHALT CONTENT	BULK SP. GRAVITY	HEIGHT (inch)	ADJUSTMENT FACTOR	STABILITY (lbs.)	ADJUSTED STABILITY	FLOW (.01 in.)
1	4.53	2.471	2.482	0	2275	2275	13
2	4.53	2.492	2.473	0	2450	2450	13
3	4.53	2.479	2.477	0	2450	2450	12
AVERAGE	4.53	2.481	2.477			2392	12.667
			1/2 Inch GRADATION				
1	4.95	2.465	2.507	0	2000	2000	13
2	4.95	2.48	2.529	0	2025	2025	12
3	4.95	2.509	2.495	0	2365	2365	13
AVERAGE	4.95	2.485	2.510			2130	12.667
			3/4 Inch GRADATION				
1	4.25	2.473	2.451	1.04	1750	1820	12
2	4.25	2.516	2.475	0	2150	2150	13
3	4.25	2.505	2.503	0	2162	2162	15
AVERAGE	4.25	2.498	2.476			2044	13.333
			1 Inch GRADATION				
1	3.82	2.526	2.475	0	2088	2088	13
2	3.82	2.532	2.471	0	2513	2513	14.5
3	3.82	2.53	2.471	0	2188	2188	13
AVERAGE	3.82	2.529	2.472			2263	13.500
			1 1/2 Inch GRADATION				
1	3.37	2.535	2.512	0	2000	2000	14.5
2	3.37	2.531	2.509	0	2075	2075	16
3	3.37	2.549	2.455	1.04	2525	2626	15.5
AVERAGE	3.37	2.538	2.492			2234	15.333

Table 23

4 Inch Diameter Creep Test Results

MAX. AGG. SIZE	SPEC. GRAV.	SAMPLE HT. (in.)	MAX. DEFOR. (in.)	REBOUND (in.)	PERM. DEFOR. (in.)
3/8	2.493	2.476	0.0139	0.0025	0.0115
3/8	2.490	2.479	0.0127	0.0029	0.0098
3/8	2.494	2.486	0.0105	0.0024	0.0080
Avg.			0.0124	0.0026	0.0098
1/2	2.503	2.518	0.0146	0.0024	0.0122
1/2	2.502	2.504	0.0128	0.0025	0.0102
1/2	2.514	2.505	0.0141	0.0025	0.0116
Avg.			0.0138	0.0025	0.0114
3/4	2.534	2.488	0.0215	0.0023	0.0192
3/4	2.481	2.525	0.0113	0.0017	0.0096
3/4	2.512	2.468	0.0133	0.0021	0.0112
Avg.			0.0154	0.0020	0.0133
1	2.521	2.472	0.0127	0.0020	0.0106
1	2.538	2.464	0.0131	0.0017	0.0114
1	2.533	2.485	0.0150	0.0024	0.0127
Avg.			0.0136	0.0020	0.0116
1 1/2	2.549	2.474	0.0087	0.0021	0.0065
1 1/2	2.530	2.476	0.0158	0.0016	0.0142
1 1/2	2.535	2.470	0.0293	0.0019	0.0275
Avg.			0.0179	0.0019	0.0161

Table 24

6 Inch Diameter Creep Test Results

MAX. AGG. SIZE	SPEC. GRAV.	SAMPLE HT. (in.)	MAX. DEFOR. (in.)	REBOUND (in.)	PERM. DEFOR. (in.)
3/8	2.480	3.763	0.0221	0.0038	0.0183
3/8	2.479	3.720	0.0198	0.0042	0.0156
3/8	2.473	3.751	0.0172	0.0034	0.0138
Avg.			0.0197	0.0038	0.0159
1/2	2.509	3.714	0.0247	0.0039	0.0208
1/2	2.503	3.729	0.0239	0.0046	0.0193
1/2	2.482	3.732	0.0211	0.0039	0.0171
Avg.			0.0232	0.0041	0.0191
3/4	2.511	3.699	0.0276	0.0045	0.0231
3/4	2.496	3.683	0.0261	0.0040	0.0221
3/4	2.519	3.689	0.0198	0.0037	0.0160
Avg.			0.0245	0.0041	0.0204
1	2.536	3.688	0.0195	0.0039	0.0156
1	2.545	3.686	0.0188	0.0032	0.0156
1	2.540	3.678	0.0203	0.0040	0.0163
Avg.			0.0195	0.0037	0.0158
1 1/2	2.564	3.699	0.0181	0.0035	0.0146
1 1/2	2.554	3.700	0.0180	0.0039	0.0141
1 1/2	2.559	3.663	0.0173	0.0038	0.0135
Avg.			0.0178	0.0037	0.0141

Table 25

4 Inch and 6 Inch Diameter Indirect Tensile Test Results

MAXIMUM AGGREGATE SIZE	ASPHALT CON. (%)	SPECIMEN HEIGHT 6"/4"	INDIRECT TENSILE LOAD (6")	IND.TEN. STRESS (6" psi)	AVERAGE IN. TEN. STR. (6") LOAD/St	INDIRECT TENSILE LOAD (4")	IND. TEN. STRESS (4" psi)	AVERAGE IN. TEN. STR. (4") LOAD/St
3/8"	4.53	3.702	4100.00	117.50				
3/8"		2.471				2200.00	141.68	
3/8"	4.53	3.674	4225.00	122.00	4233.33			
3/8"		2.488			121.45	1950.00	124.72	2125.00
3/8"	4.53	3.718	4375.00	124.84				136.03
3/8"		2.499				2225.00	141.69	
1/2"	4.95	3.714	3800.00	108.55				
1/2"		2.507				2125.00	134.89	
1/2"	4.95	3.720	3925.00	111.94	3891.67			
1/2"		2.496			111.15	2200.00	140.26	2175.00
1/2"	4.95	3.709	3950.00	112.98				138.52
1/2"		2.493				2200.00	140.43	
3/4"	4.25	3.723	3725.00	106.15				
3/4"		2.468				2450.00	157.97	
3/4"	4.25	3.720	3825.00	109.08	3800.00			
3/4"		2.476			108.55	2500.00	160.68	2416.67
3/4"	4.25	3.699	3850.00	110.42				155.47
3/4"		2.477				2300.00	147.76	
1"	3.82	3.697	4200.00	120.52				
1"		2.462				2125.00	137.35	
1"	3.82	3.665	4100.00	118.68	3991.67			
1"		2.471			114.66	2175.00	140.07	2100.00
1"	3.82	3.718	3675.00	104.86				135.42
1"		2.470				2000.00	128.85	
1 1/2"	3.37	3.697	4275.00	122.68				
1 1/2"		2.467				1662.50	107.24	
1 1/2"	3.37	3.710	4325.00	123.68	4258.33			
1 1/2"		2.462			121.94	2350.00	151.90	2195.83
1 1/2"	3.37	3.707	4175.00	119.48				141.74
1 1/2"		2.467				2575.00	166.10	

Table 26

3/8 Inch Mix Resilient Modulus Test Results

Asphalt Content = 4.53 %　　　　Load duration = 0.10 sec.
Ave. Ind. Tens. Str. = 136.048 p.s.iLd. Frequency = 1.0 sec.
Poisson's Ratio = 0.35

TEST NO.	% OF St	LOAD (lbs.)	TEMP. (deg. F.)	HEIGHT (in.)	CHANGE IN HEIGHT (0,90) (E -6 in)	AVE. CHG. IN HT. (E -6 IN)	RESILIENT MODULUS	AVERAGE RESILIENT MODULUS
1	10	212	41	2.475	25/25	25	2124283	
1	10	212	77	2.475	37.5/50	44	1213876	
1	10	212	104	2.475	800/300	550	96558	
2	10	212	41	2.476	18.75/25	22	2426771	2458371
2	10	212	77	2.476	37.5/37.5	38	1415617	1229505
2	10	212	104	2.476	550/500	525	101115	101192
3	10	213	41	2.494	18.75/18.75	19	2824058	
3	10	213	77	2.494	50/50	50	1059022	
3	10	213	104	2.494	550/450	500	105902	
1	15	317	41	2.475	50/50	50	1588202	
1	15	317	77	2.475	62.5/62.5	63	1270562	
1	15	317	104	2.475	750/550	650	122169	
2	15	318	41	2.476	37.5/37.5	38	2123425	1944328
2	15	318	77	2.476	62.5/75	69	1158232	1195299
2	15	318	104	2.476	1800/550	1175	67769	90510
3	15	320	41	2.494	37.5/37.5	38	2121358	
3	15	320	77	2.494	62.5/75	69	1157104	
3	15	320	104	2.494	1050/900	975	81591	
1	20	424	41	2.475	50/62.5	56	1888251	
1	20	424	77	2.475	93.75/100	97	1096404	
1	20	424	104	2.475	1200/1000	1100	96558	
2	20	424	41	2.476	56.25/62.5	59	1788147	1931481
2	20	424	77	2.476	100/100	100	1061712	1095913
2	20	424	104	2.476	1300/1250	1275	83272	90977
3	20	426	41	2.494	50/50	50	2118043	
3	20	426	77	2.494	87.5/100	94	1129623	
3	20	426	104	2.494	875/1400	1138	93101	

Table 27

1/2 Inch Mix Resilient Modulus Test Results

Asphalt Content = 4.95 %
Ave. Ind. Tens. Str. = 138.545 p.s.i.

Load Duration = 0.10 sec.
Ld. Frequency = 1.0 sec.
Poisson's Ratio = 0.35

TEST NO.	% OF St	LOAD (lbs.)	TEMP. (deg. F.)	HEIGHT (in.)	CHANGE IN HEIGHT (0,90) (E -6 IN)	AVE. CHG. IN HT. (E -6 IN)	RESILEINT MODULUS	AVERAGE RESILEINT MODULUS
1	10	218	41	2.503	30/33	32	1714260	
1	10	218	77	2.503	100/130	115	469558	
1	10	218	104	2.503	1000/1150	1075	50232	
2	10	217	41	2.496	21/27	24	2245927	1951632
2	10	217	77	2.496	120/130	125	431218	463893
2	10	217	104	2.496	1900/700	1300	41463	43422
3	10	218	41	2.503	21/36	29	1894709	
3	10	218	77	2.503	100/120	110	490902	
3	10	218	104	2.503	900/1900	1400	38571	
1	15	327	41	2.503	42/54	48	1687475	
1	15	327	77	2.503	180/280	230	352169	
1	15	327	104	2.503	2200/2300	2250	35999	
2	15	326	41	2.496	42/54	48	1687033	1767684
2	15	326	77	2.496	260/240	250	323910	363819
2	15	326	104	2.496	1500/1600	1550	52244	39414
3	15	327	41	2.503	42/42	42	1928543	
3	15	327	77	2.503	160/230	195	415378	
3	15	327	104	2.503	1800/3600	2700	30000	
1	20	436	41	2.503	66/72	69	1565194	
1	20	436	77	2.503	240/270	255	423523	
1	20	436	104	2.503	1500/2200	1850	58378	
2	20	435	41	2.496	72/72	72	1500735	1501969
2	20	435	77	2.496	310/330	320	337665	380043
2	20	435	104	2.496	2200/2400	2300	46980	56295
3	20	436	41	2.503	72/78	75	1439979	
3	20	436	77	2.503	270/300	285	378942	
3	20	436	104	2.503	1700/1700	1700	63528	

Table 28

3/4 Inch Mix Resilient Modulus Test Results

Asphalt Content = 4.25 %   Load Duration = 0.10 sec.
Ave. Ind. Tens. Str. = 155.491 p.s.i. Ld. Frequency = 1.0 sec.
Poisson's Ratio = 0.35

TEST NO.	% OF St	LOAD (lbs.)	TEMP. (deg. F.)	HEIGHT (in.)	CHANGE IN HEIGHT (0,90) (E -6 IN)	AVE. CHG. IN HT. (E -6 IN)	RESILIENT MODULUS	AVERAGE RESILIENT MODULUS
1	10	241	41	2.485	30/30	30	2004292	
1	10	241	77	2.485	280/240	260	231265	
1	10	241	104	2.485	825/500	663	90760	
2	10	242	41	2.467	30/30	30	2027294	2016355
2	10	242	77	2.467	240/310	275	221159	219197
2	10	242	104	2.467	750/1500	1125	54061	60883
3	10	242	41	2.479	30/30	30	2017480	
3	10	242	77	2.479	230/360	295	205167	
3	10	242	104	2.479	1850/1350	1600	37828	
1	15	362	41	2.485	50/50	50	1806358	
1	15	362	77	2.485	260/280	270	334511	
1	15	362	104	2.485	1600/1200	1400	64513	
2	15	363	41	2.467	70/70	70	1303260	1440035
2	15	363	77	2.467	230/280	255	357758	344953
2	15	363	104	2.467	1600/1400	1500	60819	64617
3	15	363	41	2.479	70/80	75	1210488	
3	15	363	77	2.479	250/280	265	342591	
3	15	363	104	2.479	1100/1550	1325	68518	
1	20	482	41	2.485	80/80	80	1503219	
1	20	482	77	2.485	310/350	330	364417	
1	20	482	104	2.485	1900/*	1900	63293	
2	20	484	41	2.467	110/130	120	1013647	1223237
2	20	484	77	2.467	360/450	405	300340	313813
2	20	484	104	2.467	1800/3400	2600	46784	49922
3	20	484	41	2.479	100/110	105	1152846	
3	20	484	77	2.479	475/400	438	276683	
3	20	484	104	2.479	3100/3000	3050	39688	

* Sample failed underoad.

Table 29

1 Inch Mix Resilient Modulus Test Results

Asphalt Content = 3.82 %
Ave. Ind. Tens. Str. = 135.44 p.s.i.
Load Duration = 0.10 sec.
Ld. Frequency = 1.0 sec.
Poisson's Ratio = 0.35

TEST NO.	% OF St	LOAD (lbs.)	TEMP. (deg. F.)	HEIGHT (in.)	CHANGE IN HEIGHT (0,90) (E -6 IN)	AVE. CHG. IN HT. (E -6IN)	RESILIENT MODULUS	AVERAGE RESILIENT MODULUS
1	10	210	41	2.462	21/30	26	2073876	
1	10	210	77	2.462	80/120	100	528838	
1	10	210	104	2.462	700/1350	1025	51594	
2	10	211	41	2.481	27/30	29	1850131	1960359
2	10	211	77	2.481	90/90	90	585875	531695
2	10	211	104	2.481	1300/1350	1325	39795	44842
3	10	210	41	2.464	24/30	27	1957071	
3	10	210	77	2.464	110/110	110	480372	
3	10	210	104	2.464	1100/1350	1225	43135	
1	15	314	41	2.462	36/48	42	1882712	
1	15	314	77	2.462	170/210	190	416179	
1	15	314	104	2.462	2500/2900	2700	29287	
2	15	317	41	2.481	42/42	42	1886144	1790032
2	15	317	77	2.481	190/170	180	440100	424481
2	15	317	104	2.481	1800/1200	1500	52812	38609
3	15	315	41	2.464	48/51	50	1601240	
3	15	315	77	2.464	200/180	190	417165	
3	15	315	104	2.464	1600/3100	2350	33728	
1	20	419	41	2.462	60/66	63	1674855	
1	20	419	77	2.462	250/260	255	413788	
1	20	419	104	2.462	*/*	*	*	
2	20	422	41	2.481	54/66	60	1757625	1594716
2	20	422	77	2.481	270/240	255	413559	394912
2	20	422	104	2.481	2000/2500	2250	46870	*
3	20	419	41	2.464	84/72	78	1351669	
3	20	419	77	2.464	250/340	295	357390	
3	20	419	104	2.464	2300/5200	3750	28115	

* Sample failed under load.

Table 30

1 1/2 Inch Mix Resilient Modulus Test Results

Asphalt Content = 3.37 % Load Duration = 0.10 sec.
Ave. Ind. Tens. Str. = 141.764 p.s.i. Ld. Frequency = 1.0 sec.
Poisson's Ratio = 0.35

TEST NO.	% OF St	LOAD (lbs.)	TEMP. (deg. F.)	HEIGHT (in.)	CHANGE IN HEIGHT (0,90) (E -6 IN)	AVE. CHG. IN HT. (E -6 IN)	RESILIENT MODULUS	AVERAGE RESILIENT MODULUS
1	10	219	41	2.454	20/22.5	21.3	2603768	
1	10	219	77	2.454	50/60	55.0	1006001	
1	10	219	104	2.454	600/300	450.0	122956	
2	10	218	41	2.448	15/35	25.0	2208497	2421973
2	10	218	77	2.448	75/70	72.5	761551	782894
2	10	218	104	2.448	600/650	625.0	88340	96721
3	10	217	41	2.437	20/25	22.5	2453654	
3	10	217	77	2.437	90/100	95.0	581129	
3	10	217	104	2.437	850/550	700.0	78867	
1	15	328	41	2.454	30/40	35.0	2367680	
1	15	328	77	2.454	100/115	107.5	770872	
1	15	328	104	2.454	1200/650	925.0	89588	
2	15	327	41	2.448	30/35	32.5	2548265	2428534
2	15	327	77	2.448	125/140	132.5	625046	670092
2	15	327	104	2.448	1500/950	1225.0	67607	83994
3	15	326	41	2.437	40/30	35.0	2369658	
3	15	326	77	2.437	130/140	135.0	614356	
3	15	326	104	2.437	900/850	875.0	94786	
1	20	437	41	2.454	40/35	37.5	2944200	
1	20	437	77	2.454	160/180	170.0	649456	
1	20	437	104	2.454	1250/1600	1425.0	77479	
2	20	436	41	2.448	45/35	40.0	2760621	2821727
2	20	436	77	2.448	180/220	200.0	552124	560548
2	20	436	104	2.448	1400/1050	1225.0	90143	81257
3	20	434	41	2.437	40/40	40.0	2760361	
3	20	434	77	2.437	220/440	230.0	480063	
3	20	434	104	2.437	1450/*	1450.0	76148	

* Sample failed under load.