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16. Abstract

This study was undertaken to measure what, if any, differences exist in strength between mixtures made with 3/4-in (19-mm) maximum size aggregate and those made with 1/2-in (13-mm) maximum size aggregate. In order to make a comparison, a definition of *equivalent gradation* was necessary. This definition used a constant slope, n, of the log percent passing versus the log sieve size. Tests included gyratory shear, creep, resilient modulus, indirect tensile strength, and failure strain. It was concluded that with the gradations and aggregates studied, the type of aggregate is a more significant source of difference in strength than either aggregate maximum size or the gradation used.

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FINAL REPORT

STRENGTH OF VIRGINIA'S 1/2-INCH AND 3/4-INCH ASPHALT SURFACE MIXTURES

C. S. Hughes
Research Consultant

(The opinions, findings, and conclusions expressed in this report are those of the author and not necessarily those of the sponsoring agencies.)

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ABSTRACT

This study was undertaken to measure what, if any, differences exist in strength between mixtures made with 3/4-in (19-mm) maximum size aggregate and those made with 1/2-in (13-mm) maximum size aggregate. In order to make a comparison, a definition of *equivalent gradation* was necessary. This definition used a constant slope, n, of the log percent passing versus the log sieve size. Tests included gyratory shear, creep, resilient modulus, indirect tensile strength, and failure strain. It was concluded that with the gradations and aggregates studied, the type of aggregate is a more significant source of difference in strength than either aggregate maximum size or the gradation used.

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INTRODUCTION

Increases in the number of trucks and in their tire pressures and loads have created stresses in the upper pavement layers that exceed the strength of many asphalt mixtures designed using traditional procedures. This problem, which is evidenced primarily by rutting in the surface, is generally caused by increased densification under traffic, which reduces the air voids to the point that aggregate-to-aggregate contact in the mixture is eventually reduced, thereby resulting in loss of shear strength. To counteract this problem, VDOT (and many other states) increased the compactive effort of the mixture design by using a 75-blow Marshall compactive effort rather than the traditional 50-blow compactive effort. This tends to reduce the design asphalt content by 0.3 to 0.5 percent, thus allowing for a greater amount of compaction under traffic before the air voids are reduced to an unacceptable level. However, the Marshall procedure may tend to overcompensate for traffic level. A recent study by Maupin[1] recommends that shear-strength results obtained from the Gyratory Testing Machine (GTM) be used as a supplement to the Marshall design to check mixtures with potential strength problems.

In addition to reducing the asphalt content, there are other approaches to increasing the shear strength. Some of these include adding a modifier or filler to the mix, using 100 percent crushed material, or altering the gradation. By increasing the amount and size of the maximum size aggregate, the shear strength would be expected to increase because of the larger volume of large aggregate resisting the shear stresses. There are two standard surface mixtures in Virginia: SM-2, which allows up to 3 percent of the aggregate to be retained on the 1/2-in (13-mm) sieve, and SM-3, which allows up to 3 percent to be retained on the 3/4-in (19-mm) sieve. The SM-3 has been used for roads with high traffic under the assumption that it is a fundamentally stronger mixture than the SM-2 mixture. However, specific differences in strength between the two mixture types have not been established, and the conditions of gradation under which one might be preferable to the other have not been well defined. When comparing mixtures with different maximum size aggregate, it is important to establish gradations that are considered equivalent to avoid biases that may be created by arbitrarily selecting gradations.

Although optimization of strength is a primary concern, consideration must be given to other aspects of good performance, such as, durability, skid resistance, and the ride quality of the surface. It has been observed that within the designated

VDOT gradation bands for surface mixtures, very different surface textures can be obtained. This is particularly true for the 1/2-in (13-mm) maximum size aggregate mixture, which allows from 6 to 18 percent of the aggregate to be retained on the 3/8-in (10-mm) sieve. Those mixtures containing close to the minimum percentage retained on this sieve have a significantly different texture from those having close to the maximum percentage retained. The finer textured mixtures, i.e., those with the lower amounts of 3/8-in (10-mm) material retained, generally tend to have lower skid resistance and greater loss of skid number with speed than those with the coarser texture.[2] Thus, there is a need, not only to compare differences between present 1/2-in (13-mm) and 3/4-in (19-mm) mixture gradation bands but also to quantify differences obtained within the gradation limits now permitted within each type.

PURPOSE

The purposes of this study were

1. to review the principles for selecting the gradations of asphalt mixtures with different maximum size aggregates including the principles recommended for selecting voids in the mineral aggregate (VMA) and voids total mixture (VTM) and to establish criteria for equivalent gradations of different size aggregates

2. to compare the overall strength of a 1/2-in (13-mm) maximum size aggregate mixture with that of a 3/4-in (19-mm) maximum size aggregate mixture

3. to establish specification requirements for the satisfactory performance of the 1/2-in (13-mm) and 3/4-in (19-mm) mixtures.

METHODS

Review of Literature

A literature review was conducted to determine the measures of gradation that would provide equivalent gradations for 1/2-in (13-mm) and 3/4-in (19-mm) maximum size aggregates. Numerous studies have been made to establish the proper grading of the aggregate for bituminous mixtures. The studies date at least as far back as 1907 when Fuller and Thompson published recommendations for proportioning aggregate in portland cement concrete.[3] In 1948, Nijboer reviewed existing concepts and developed additional theory concerning the design of dense bituminous "road carpets."[4] Based on Nijboer's use of a double logarithmic chart (log percentage passing versus log of sieve opening in microns), Goode and Lufsey developed the Bureau of Public Roads' (BPR) gradation chart in 1962.[5] Nijboer showed that when various gradations of the same maximum size aggregate were plotted as straight lines on the log/log chart with different slopes, that gradation with a 0.45 slope had the minimum void content, i.e., the maximum density. Goode and Lufsey confirmed this with their own laboratory studies and constructed a maximum density chart (Figure 1) in which the percentage passing is plotted on the y-axis, and the value of the sieve sizes in microns raised to the 0.45 power is plotted on the x-axis. This chart was made available to the public by the Bureau of Public Roads (now FHWA) and is still extensively used by state highway agencies and others as a means of judging the adequacy of gradations for asphalt concrete. In 1965, Hudson and Davis reviewed the development of the major gradation concepts up to that time and presented an arithmetical method for computing the percentage of voids in the mineral aggregate (VMA).[6] This method involved a stepwise estimation of the reduction in voids when segments of larger aggregate were added beginning with an estimated value for voids in the filler. Factors for the reduction of voids for successive sieve sizes were developed based on the ratio of the sizes of the openings in the sieves and theoretical considerations. Differences in factors for rounded aggregate and angular aggregate were also shown.

In 1970, Lees re-examined the factors affecting the packing and porosity (voids) of aggregates and proposed a general theory for combining aggregates of various shapes and sizes in order to achieve minimum porosity.[7] His design method recognizes the importance of variation in aggregate shape, compactive effort (both during placement and under traffic), and the thickness of the section to be laid.

In 1989, the Association of Asphalt Paving Technologists held a symposium on pavement performance in which the effect of the grading of the aggregate was again reviewed by several authors and contributors to the published discussion.[8] These discussions revealed the considerable difference between theory and practice in asphalt mixture design. They also showed that many potential pitfalls exist between establishing a mixture design in the laboratory and obtaining the same proportions in mixtures produced at a plant. This brief review of the literature is not intended to be exhaustive, but it does provide a general picture of the development of criteria for the gradation of aggregates.

Virtually all authors state that the maximum density line for gradations does not represent the best gradation for a bituminous mixture. The VMA must be sufficiently high so that there is room for enough asphalt to provide waterproofing and durability plus a limited number of air voids to avoid permanent deformation (rutting) of the asphalt mixture. However, the maximum density line provides a reference against which a gradation can be compared to estimate whether the VMA will be adequate.

Both the shape and size of the aggregates have an important bearing on the compactability of a bituminous mixture in the laboratory, during placement, and under subsequent traffic. Thus, a mixture design must consider the compactive effort that will be imparted by traffic to maintain the necessary air voids and must

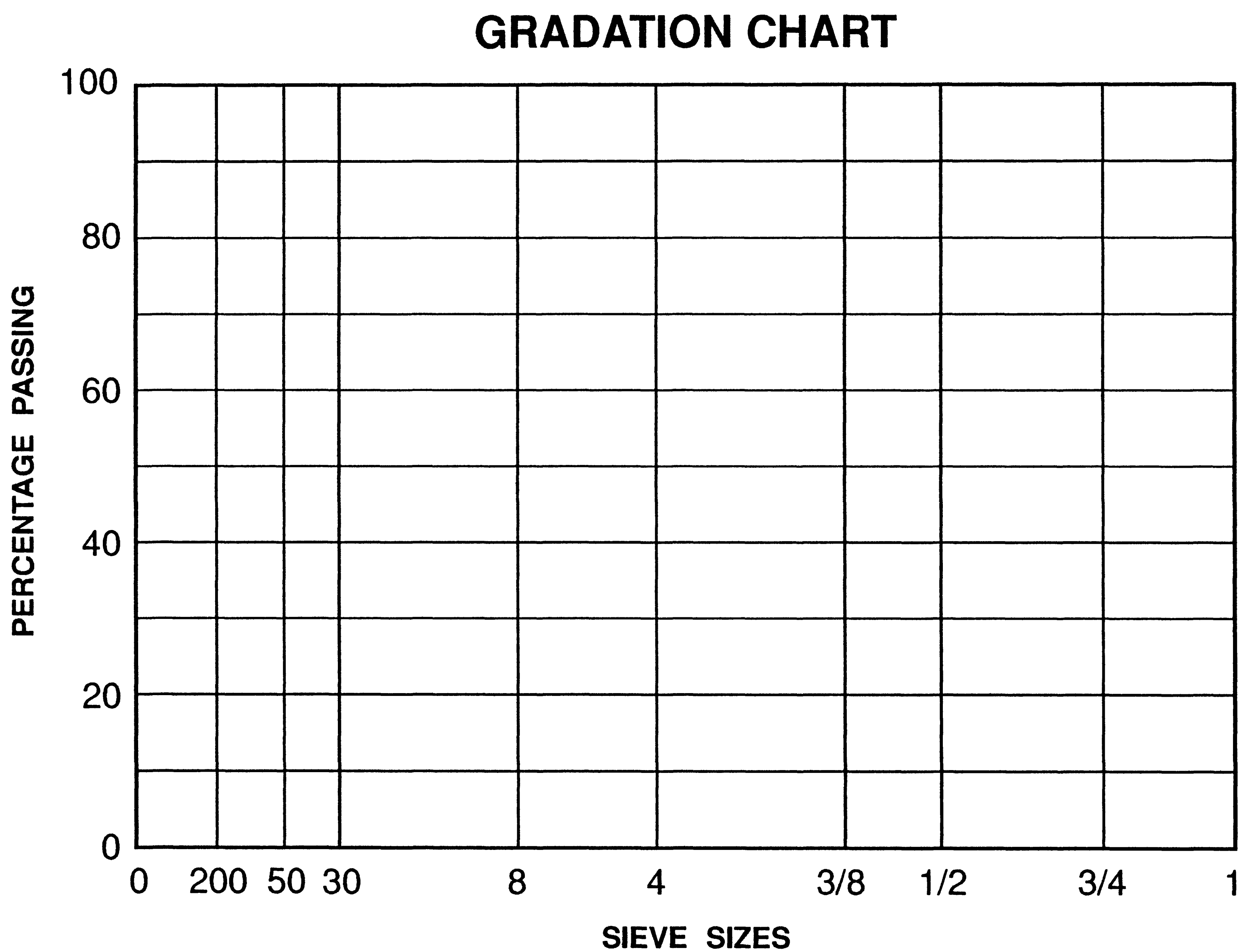


Figure 1. Maximum density gradation chart.

also consider the largest aggregate size when selecting the size of specimen to compact.

The sampling of aggregate stockpiles and the manner of preparation of the aggregates for test (dry or wet gradations for passing No. 200 [75 μm], etc.) can affect results. Accordingly, laboratory design proportions are not always duplicated by plant production. This in turn may affect optimum asphalt content and voids in the compacted mixture.

It is the aim of the design procedure to provide a mixture that will be compacted in the field sufficiently to provide the needed strength and proper asphalt content. However, it must not compact over the long-term to a point that the pavement becomes unstable because of insufficient voids in the mixture.

The amount, speed, and weight of traffic on a pavement affects the pavement's performance. It is well known that pavements that perform well under high-speed traffic for long periods of time may fail rapidly when essentially the same traffic is slowed because of some disruption of traffic flow.

Equal VMA is a reasonable measure of equivalent gradation between aggregates of the same maximum size. However, because of differences in aggregate packing, equal VMA does not provide equivalent gradations for aggregates with different maximum sizes. Following a similar approach to that taken by Brown et al.,[9] it was concluded that estimates of equivalent gradations for different maximum size aggregates are best represented by straight lines with different slopes on the FHWA gradation chart (0.45 power curve). However, these may not represent optimum gradation values for use in designing asphalt mixtures.

Although the true significance of changing the slope (n) of the log-percentage-passing-versus-log-sieve-size curve is elusive, it does appear to represent a means of estimating comparative deviations from the maximum density lines for different maximum size aggregates; thus, it may be considered a measure of equivalent gradations. To represent comparisons of equivalent gradations, a gradation representing a slope of 0.4 on the percentage passing versus log sieve size for each maximum size aggregate was chosen for this study to represent the fine side of the maximum density line, and, similarly, a slope of 0.6 was chosen to represent equivalent gradations on the coarse side of the maximum density line. A third gradation for this study was obtained by using the middle of the band recommended in ASTM Specification D-3515. When plotted on the 0.45 power chart, this provided a line with a slope of 0.53.

Experimental Design

The study included two types of aggregate typically used in asphalt surface mixtures in Virginia: granite (Manassas Stone Quarry) and quartzite (West Gravel-Grottoes). The three equivalent gradations mentioned previously for each of the 1/2-in (13-mm) and 3/4-in (19-mm) maximum size aggregates were fine (n = 0.4), medium (n = 0.53) from ASTM, and coarse (n = 0.6).

Figures 2, 3, and 4 show the relationship of the three n values for both 1/2-in (13-mm) and 3/4-in (19-mm) gradations. The VDOT design range was used as a basis for selecting the experimental gradations, but they did not correspond to constant n values; thus, in selecting the experimental gradations, the constant slope values were used after the VDOT maximum size criterion was met. For example, in Figure 2, the 1/2-in (13-mm) gradation uses a slope of $n = 0.4$ with 94 percent passing the 3/8-in (10-mm) sieve, which is the finest gradation allowed for that mixture size.

One difference in the approach used in this study compared with the one used by Brown et al. was that the full range of the gradation was used as opposed to subtracting the material passing the No. 200 sieve (75μm) as was done by Brown et al. Because the gradations were compared between different maximum size aggregates, it was thought that normalizing for the – No. 200 (75μm) material would possibly create a bias. In making this decision, as will be seen under the discussion of gradations, a large amount of – No. 200 (75μm) material is required in the gradations for $n = 0.4$. But because the gradations are compared at a constant n value, this did not present a problem. Saying this another way: the gradations selected by using an n value do not necessarily represent mixtures that would be placed on the road but do represent a valid basis for comparison in a laboratory study of equivalent gradations. This will be discussed more fully in the section on gradations and mixture design.

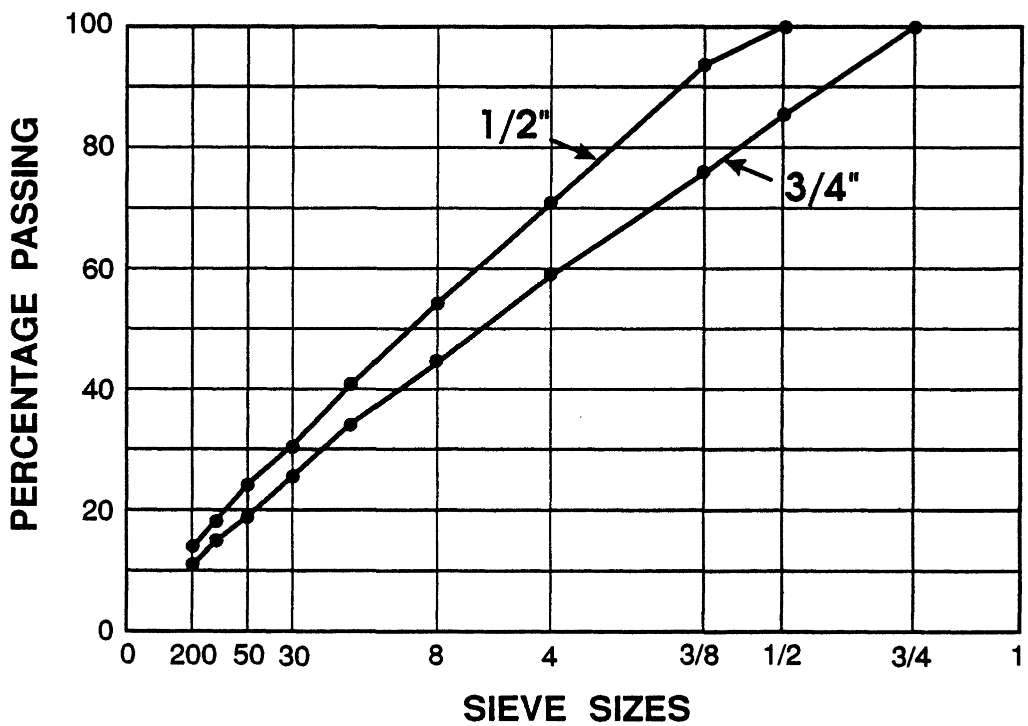


Figure 2. Gradations for $n = .4$ for 1/2-in (13-mm) and 3/4-in (19 mm) maximum size aggregates.

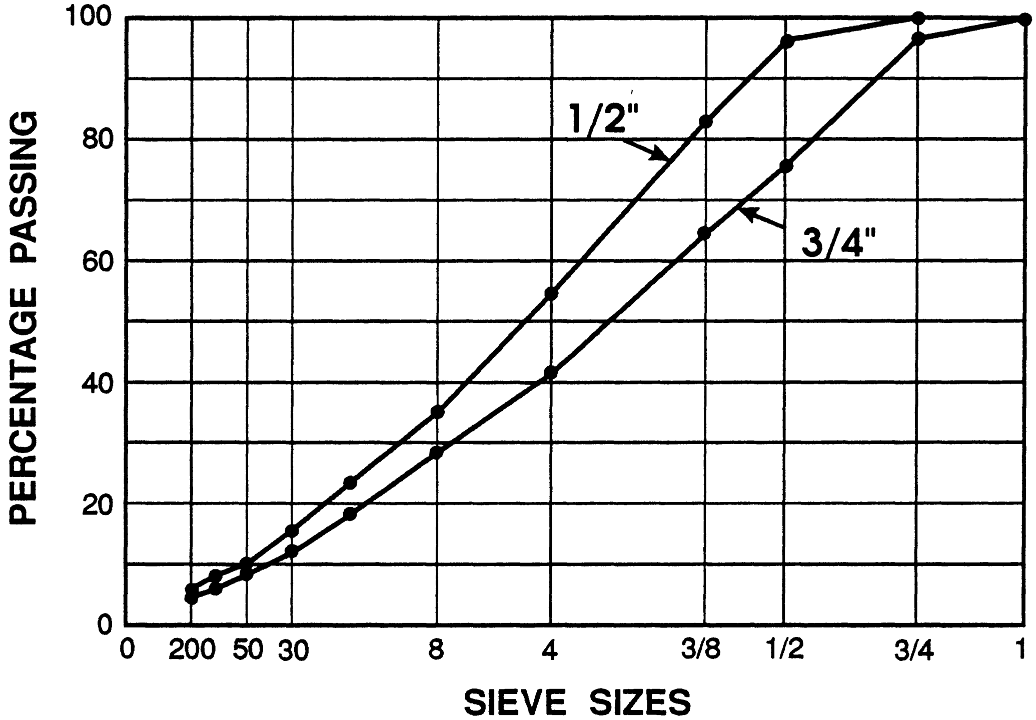


Figure 3. Gradations for n = .53 (taken from ASTM) for 1/2-in (13-mm) and 3/4-in (19 mm) maximum size aggregates.

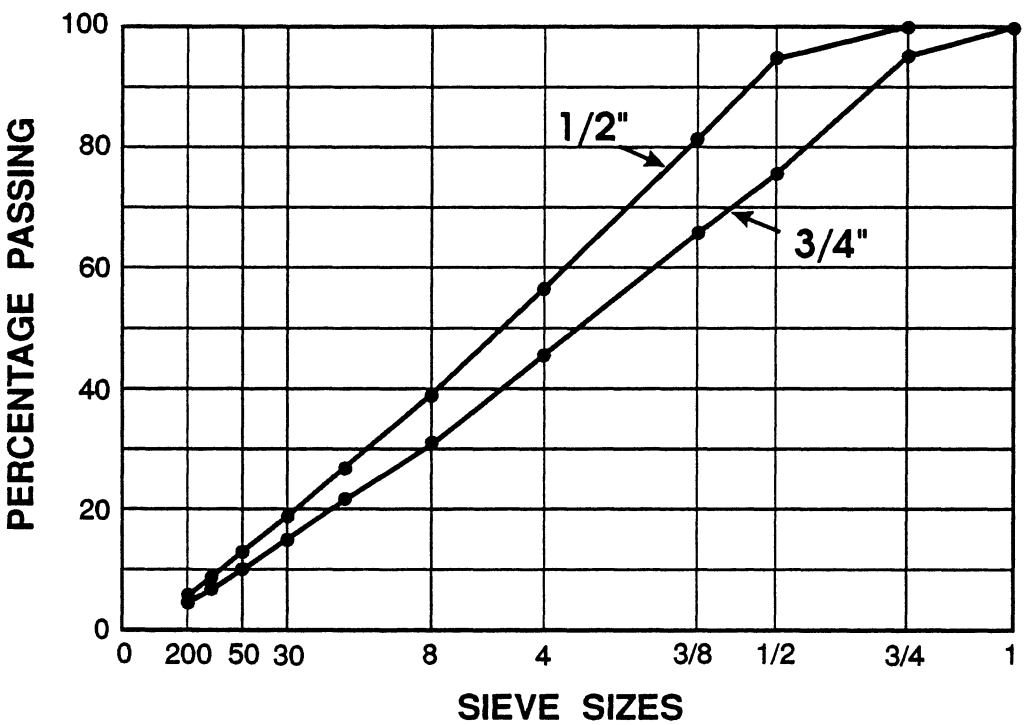


Figure 4. Gradations for n = .6 for 1/2-in (13-mm) and 3/4-in (19mm) maximum size aggregates.

For each gradation, the optimum asphalt content was determined by using the Marshall volumetric properties at a compactive effort of 75-blows.

Testing to evaluate the mixtures included using the GTM to measure shear strength, gyratory shear index, and VTM. Also, creep, resilient modulus, and indirect tensile tests were run on the mixtures to assess their strength and stiffness.

Gradations and Mixture Design

Gradations

The gradations used for each aggregate are shown in Table 1. The percentage retained on the 3/8-in (10-mm) sieve for the 1/2-in (13-mm) maximum-size mixture varies from 6 to 18 percent as allowed by the gradation bands in Virginia.

The aggregate properties are shown below:

- crushed quartzite from West Sand and Gravel (Grottoes, Virginia)
  - —effective specific gravity = 2.64
  - —bulk specific gravity = 2.57
  - —absorption = 1.05 %
  - —L.A. abrasion loss = 40.6 %

Table 1
EXPERIMENTAL GRADATIONS

Percentage Passing

	n = .4		n = .53		n = .6	
Maximum Size Aggregate	1/2 in	3/4 in	1/2 in	3/4 in	1/2 in	3/4 in
Sieve						
1"	100	100	100	100	100	100
3/4"	100	100	100	95	100	97
1/2"	100	86	95	76	97	76
3/8"	94	77	82	66	83	64
#4	71	59	57	46	55	42
#8	54	44	39	31	36	28
#16	41	34	27	22	24	18
#30	31	26	19	15	16	12
#50	24	19	13	10	10	8
#100	18	15	9	7	7	5
#200	14	11	6	5	5	4

1 in = 25.4 mm.

- granite from Vulcan Materials (Manassas, Virginia)
  - —effective specific gravity = 2.90
  - —bulk specific gravity = 2.89
  - —absorption = 0.12 %
  - —L.A. abrasion loss = 14.0 %
- natural sand from Massaponax Sand and Gravel (Massaponax, Virginia)
  - —effective specific gravity = 2.66
  - —bulk specific gravity = 2.64
  - —absorption = 0.29 %
  - —particle index = 49.36.

Mixture Designs

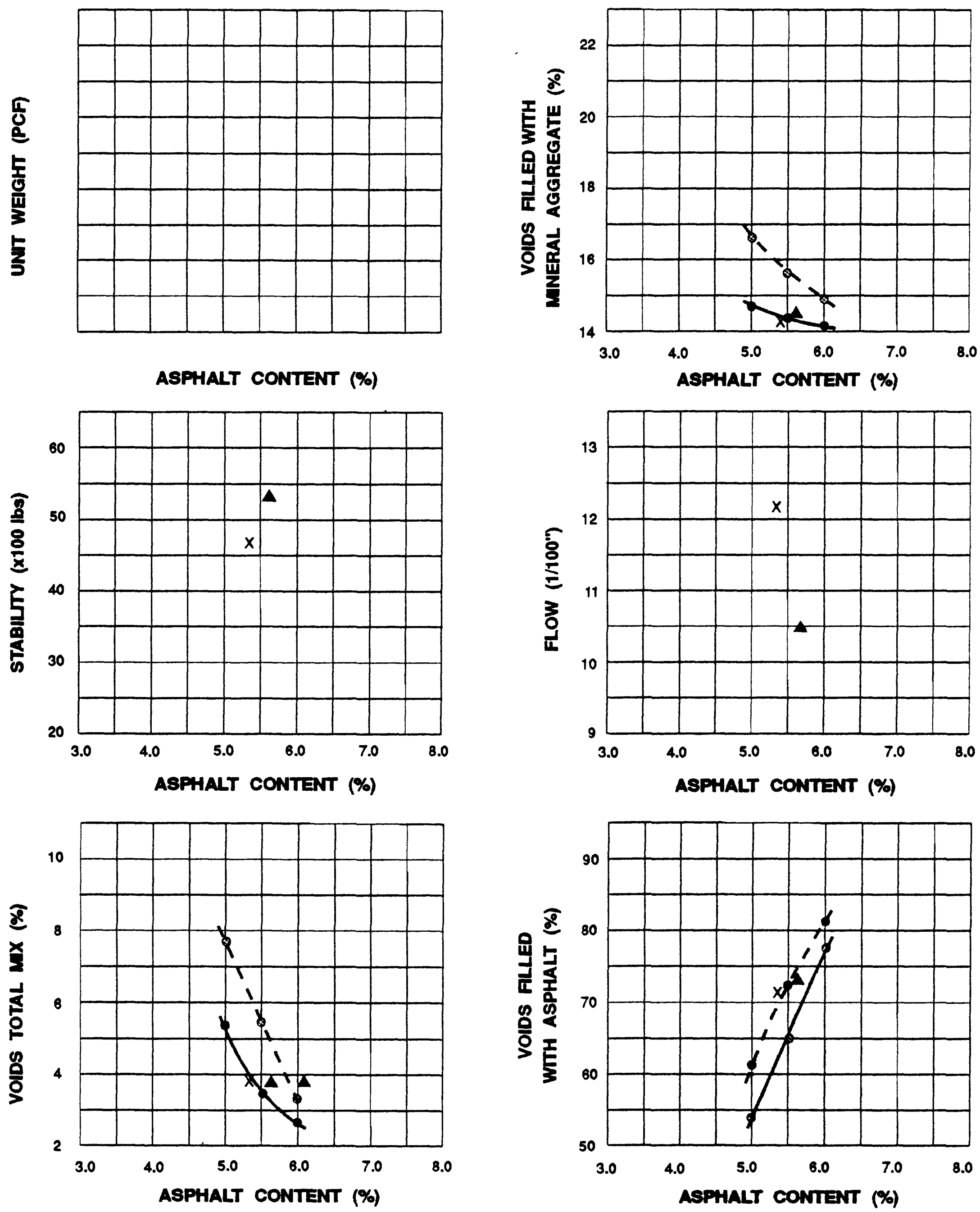
A 75-blow Marshall compactive effort was used for the mixture designs. The mixture designs for each aggregate and gradation are shown in Figures 5 through 10. Optimum asphalt contents were estimated at a VTM of 4.0 percent obtained from a mixture design procedure consisting of a minimum of three asphalt contents. A set of three specimens was then compacted at the estimated design asphalt content to confirm the results. If this set of specimens did not produce samples with VTM values 4.0 ± 0.3 percent, another set of specimens was compacted at a revised optimum asphalt content until the criteria were met. The VMA and voids filled with asphalt (VFA) values plotted are based on the aggregate bulk specific gravity. The 'x' values shown in Figures 5 through 10 indicate the revised optimum asphalt contents that were used in the remainder of the study. The mixture designs for the West quartzite were straightforward, and the optimum asphalt contents were easily determined. However, for the Manassas granite, the initial mixture design at an n value of 0.4 produced unacceptably low VTM and VMA values. It was so low that a satisfactory optimum asphalt content could not be established; thus, an additional mixture design for the fine granite mixture was made by replacing 15 percent of the fine granite with 15 percent natural sand. The replacement was done in such a manner as to maintain the same gradation as the original design. The sand had a particle index of 49.36 as determined by the National Aggregate Association's Test A.[10] Figure 8 shows mixture designs with the addition of the 15 percent sand. The VMAs for this mixture were still lower than desirable but were thought to be acceptable for comparing maximum size aggregates in this study.

This difficulty in obtaining an adequate VMA with one aggregate but not the other, even though both had exactly the same gradation indicates the differences that can occur from aggregate to aggregate as a result of particle shape, texture, or other specific aggregate properties. It is interesting to note that adding a small amount of sand that had a different particle shape than the crushed material (but did not change the amount of – No. 200 material) produced enough difference in the combined aggregate properties to substantially improve the volumetric properties of the mixture. The volumetric properties, stability, and flow at optimum asphalt content are shown in Table 2.

Contrary to expectations, the optimum asphalt content increased as the n value increased, i.e., as the mixture became coarser, a higher asphalt content was

8[illegible]

Marshall Design Charts

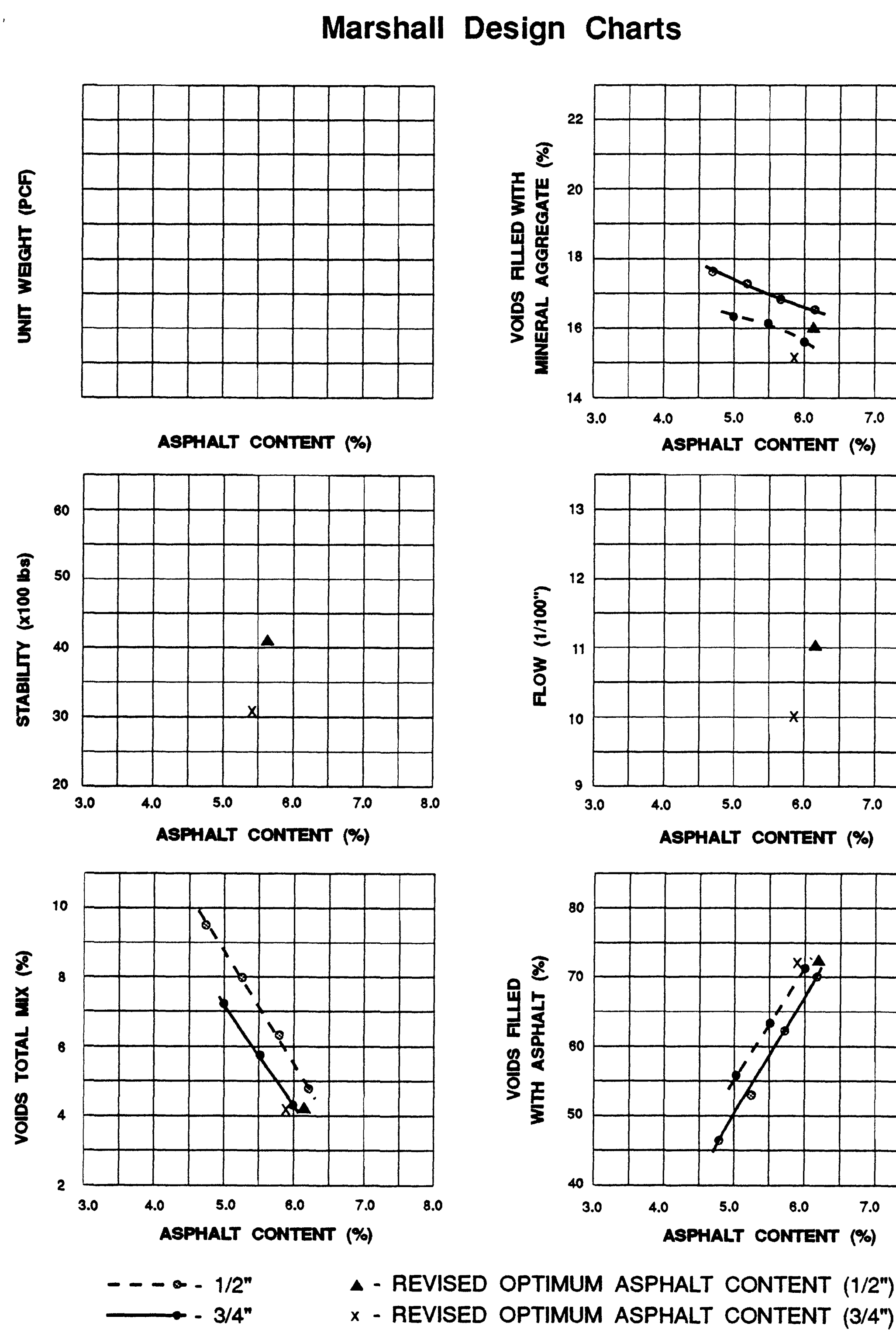


- - - - 1/2"   ▲ - REVISED OPTIMUM ASPHALT CONTENT (1/2")

——— - 3/4"   x - REVISED OPTIMUM ASPHALT CONTENT (3/4")

1 in = 25.4 mm; 1 lb = 4.45 N

Figure 5. Mixture designs with quartzite (n = .4).



1 in = 25.4 mm; 1 lb = 4.45 N

Figure 6. Mixture designs with quartzite (n = .53).

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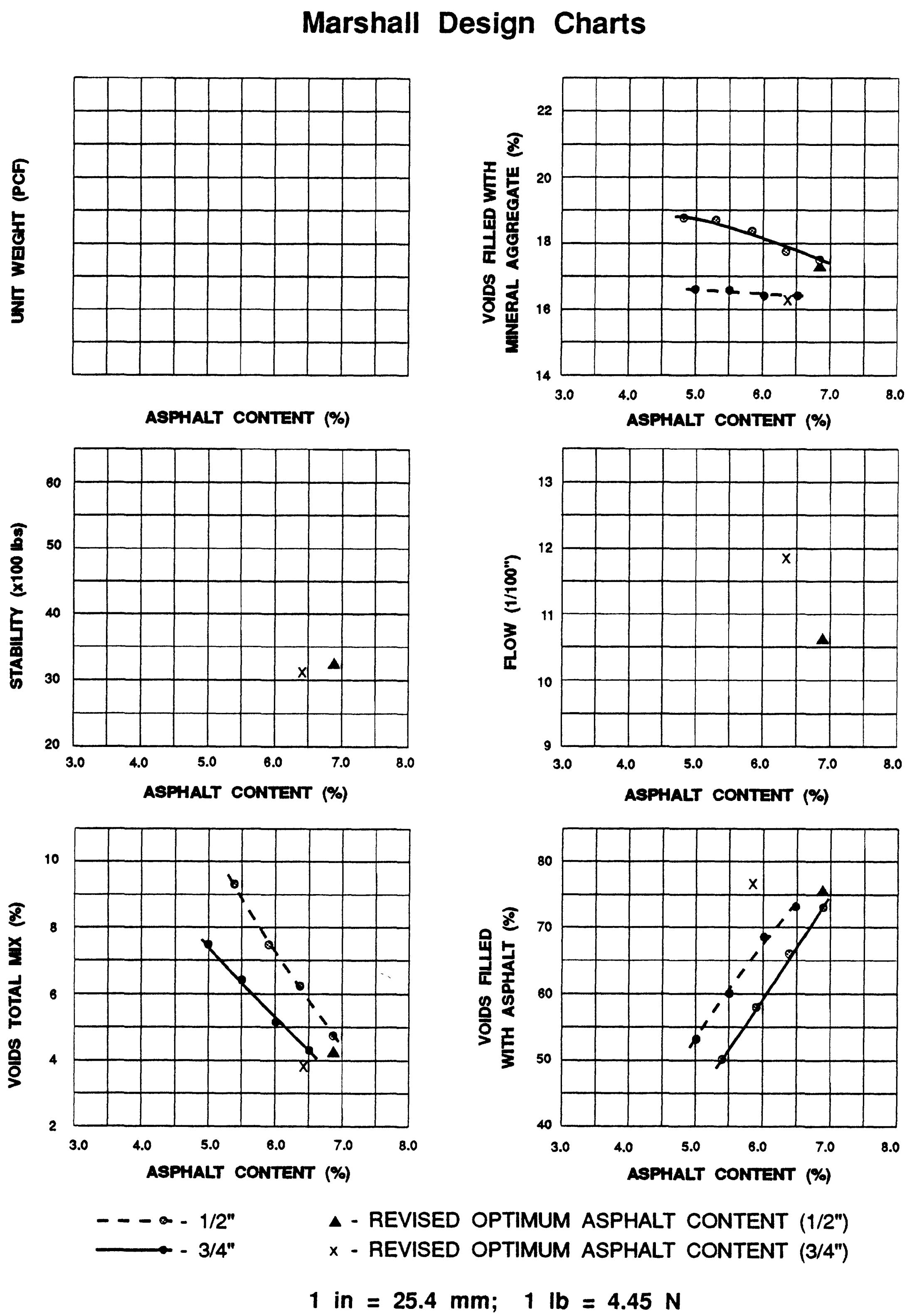


Figure 7. Mixture designs with quartzite (n = .6).

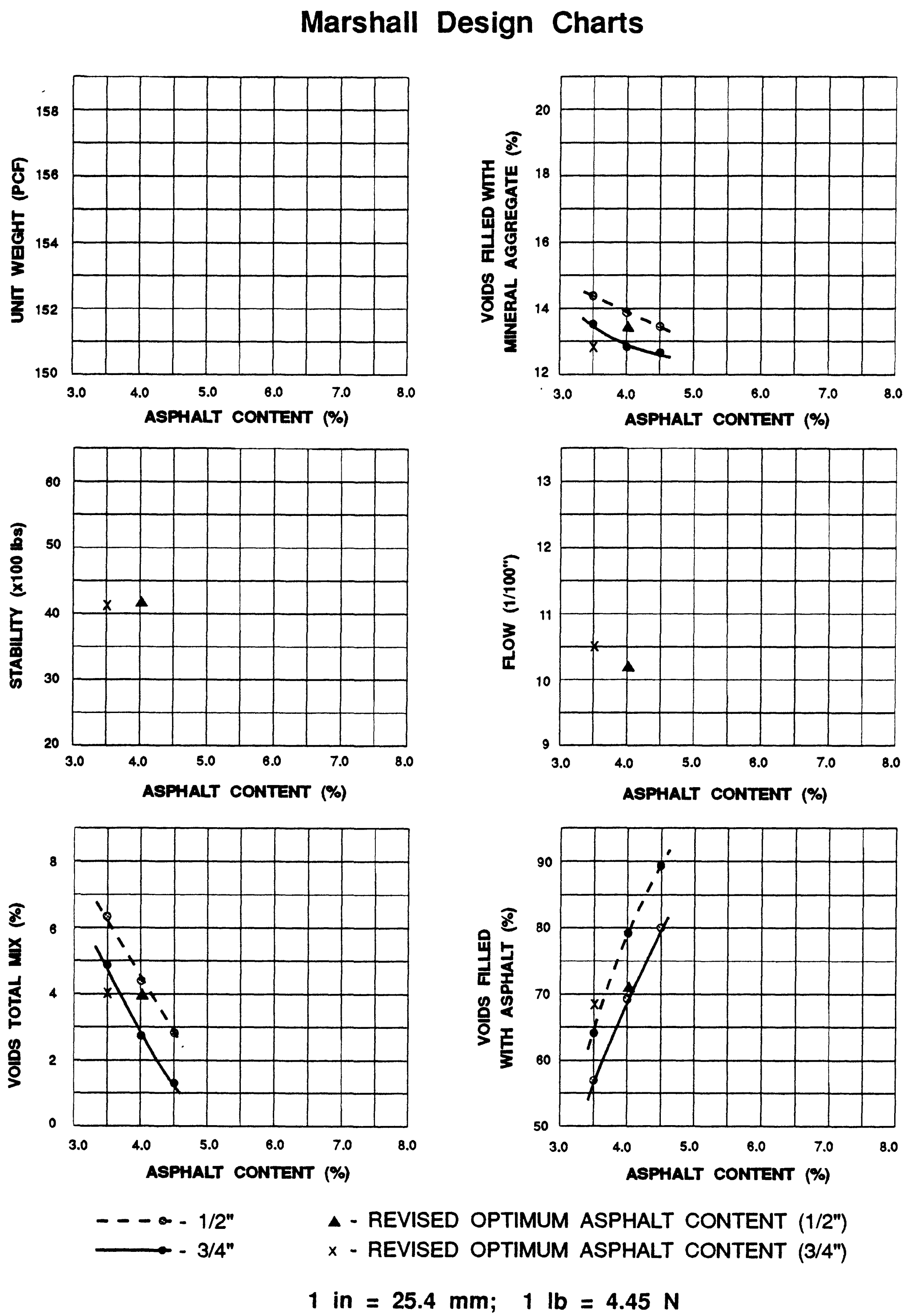
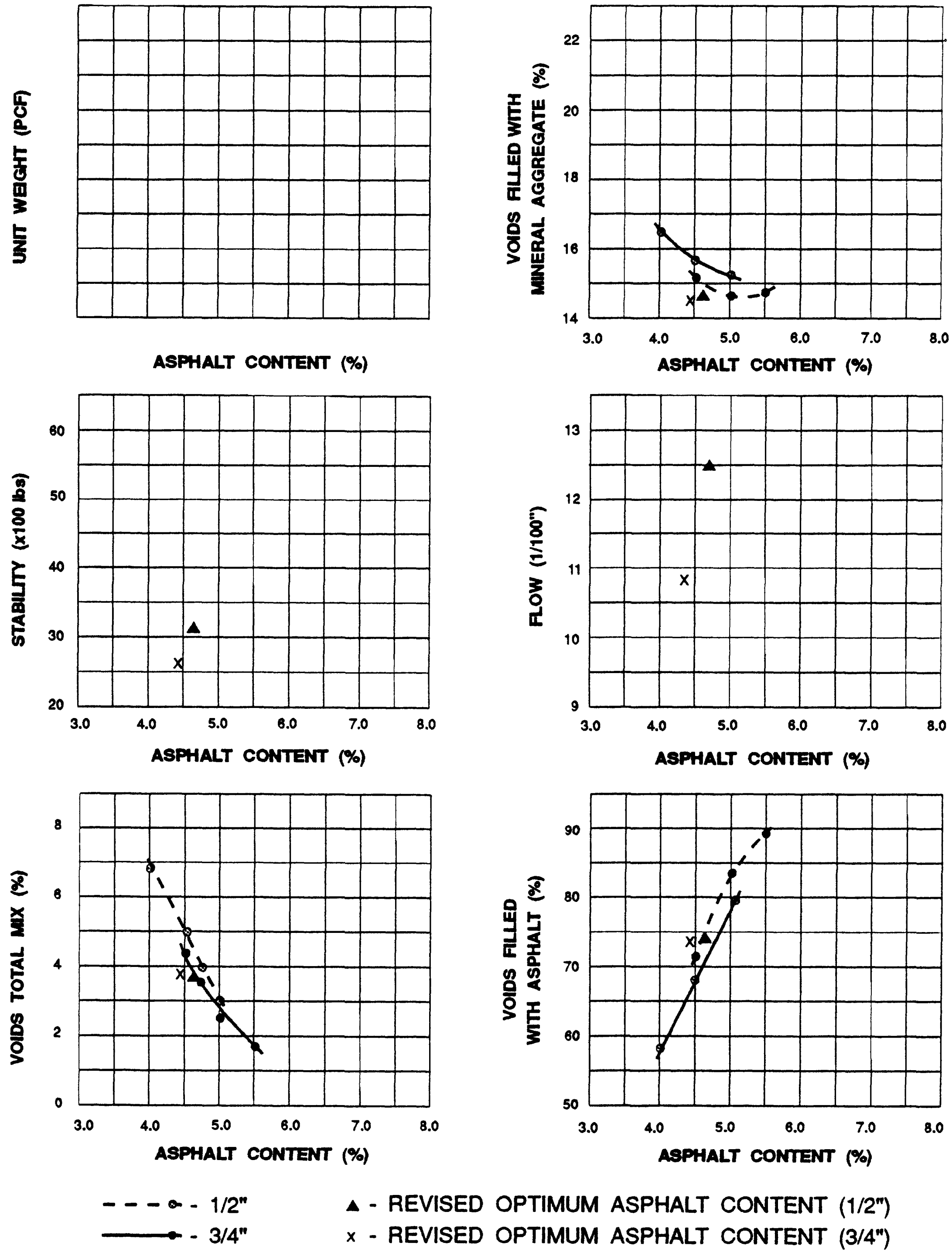


Figure 8. Mixture designs with granite (n = .4).

Marshall Design Charts



1 in = 25.4 mm; 1 lb = 4.45 N

Figure 9. Mixture designs with granite (n = .53).

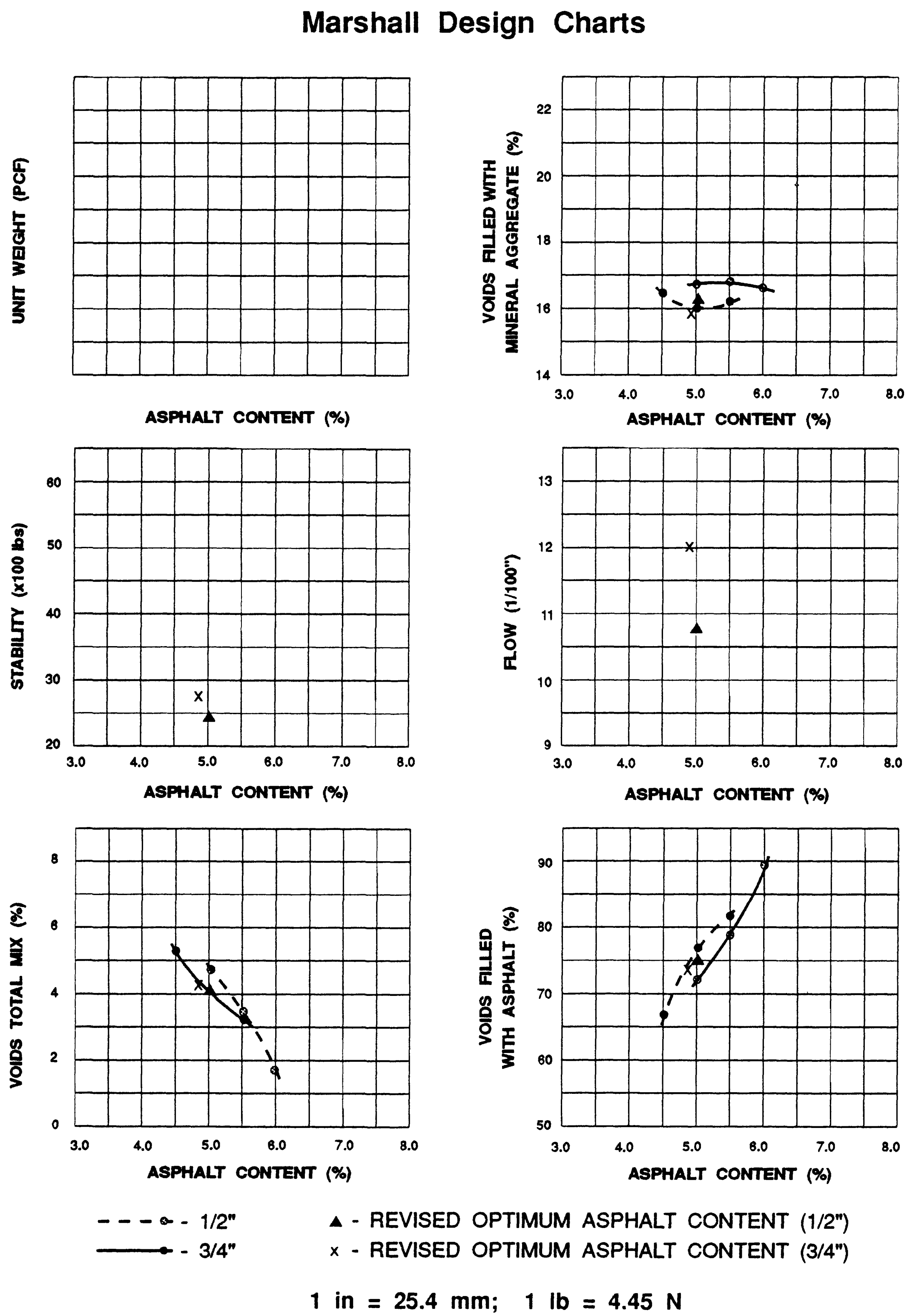


Figure 10. Mixture designs with granite (n = .6).

Table 2
MIXTURE PROPERTIES AT OPTIMUM ASPHALT CONTENT

Quartzite

n	Maximum Size Aggregate	Opt. AC	VTM	VMA Eff	VMA Bulk	VFA Eff	VFA Bulk	Stab.	Flow	F.T.*
.4	1/2 in	5.6	3.9	16.6	14.5	76.4	73.0	5333	10.5	4.7
.4	3/4 in	5.4	3.9	16.2	14.1	75.8	72.2	4700	12.3	5.4
.53	1/2 in	6.2	4.1	18.1	16.0	77.1	73.9	4127	11.0	9.6
.53	3/4 in	5.8	4.1	17.2	15.1	76.0	72.6	3113	10.0	11.0
.6	1/2 in	6.8	4.3	19.4	17.3	78.0	75.1	3240	10.7	12.8
.6	3/4 in	6.4	3.9	18.2	16.1	78.9	76.7	3080	11.8	15.1

Granite

n	Maximum Size Aggregate	Opt. AC	VTM	VMA Eff	VMA Bulk	VFA Eff	VFA Bulk	Stab.	Flow	F.T.*
.4	1/2 in	4.0	4.0	14.0	13.6	71.3	70.2	4260	10.2	3.6
.4	3/4 in	3.5	4.0	12.8	12.8	68.6	68.6	4187	10.5	3.8
.53	1/2 in	4.6	3.8	15.4	14.7	75.5	74.4	3180	12.5	6.8
.53	3/4 in	4.4	3.9	15.0	14.5	74.0	73.4	2653	10.8	7.9
.6	1/2 in	5.0	4.1	16.5	16.3	75.3	75.1	2427	10.8	8.5
.6	3/4 in	4.8	4.2	16.1	15.9	74.1	73.9	2740	12.0	10.9

*F.T. – Film thickness based on computed aggregate surface area.
1 in = 25.4 mm.

needed to reduce the voids to 4 percent. Jimenez's computer program ASPHALT,[11] which was used to analyze several mixture properties, indicated that the film thickness for a gradation of n = 0.4 was about 4 to 5 microns, for n = 0.53, the film thickness was about 7 to 11 microns, and for n = 0.6, the film thickness was about 9 to 15 microns. This meant that the fines in the finer mixtures, particularly the n = 0.4 mixture, were filling the voids and had a minimum asphalt film surrounding the aggregate. From a standpoint of durability, film thicknesses less than about 10 microns are not desirable; thus, the mixtures with n values of 0.4 would be of questionable quality.

As expected, the mixtures with the larger maximum size aggregate plot to the left of the finer mixtures on the VTM-versus-asphalt-content graph, which indicates that the former have a lower design asphalt content. Also, the slopes of the curves of the coarse and fine mixtures are essentially parallel, indicating no significant difference in sensitivity to asphalt content from a volumetric standpoint.

As indicated above, the absorption values for the two aggregates vary appreciably. Controversy exists over whether the effective or the bulk specific gravity of the aggregate should be used to calculate the volumetric properties of a mixture. If an aggregate has low absorption (e.g., the granite), there is very little difference between the volumetric properties; whereas, with the quartzite, a difference of about 2 percent exists between VMA results. Table 2 shows the calculated volumetric properties using both effective and bulk specific gravity.

RESULTS AND DISCUSSION

The results of the gyratory shear, creep, resilient modulus, and indirect-tensile-strength-and-strain-at-failure tests are discussed below. The gyratory shear, creep, and resilient modulus tests are primarily predictors of resistance to permanent deformation; whereas, the indirect-tensile-strength-and-strain-at-failure test offers a prediction of the resistance to fatigue. Test procedures and analyses for the creep and indirect-tensile-strength tests were developed by Von Quintus in the NCHRP AAMAS study.[12] All specimens were compacted by the GTM using the compaction conditions described in the next section.

Gyratory Shear Test

The GTM test conditions used were as follows: 1 degree angle of gyration, 120 psi (827 RPa) roller pressure, and a sufficient number of revolutions to achieve a compactive level such that the increase in density was 0.01 pcf (127 mg/m^3) per revolution or less. Generally, this occurred at between 150 and 180 revolutions, although with some mixtures this did not occur until 210 revolutions. The gyratory properties measured were shear strength, gyratory shear index (GSI) (an indication of the probability of a mixture approaching a plastic condition), and VTM.

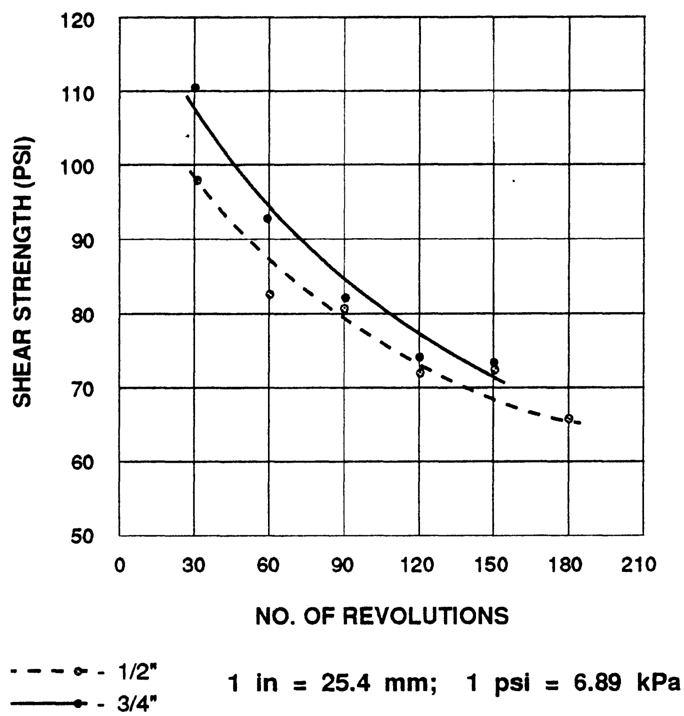


Figure 11. Shear strength v. number of revolutions (quartzite, n = .4).

A typical example of the shear-strength-versus-number-of-revolutions curve for a single n value and aggregate is shown in Figure 11 for a 1/2-in (13-mm) and a 3/4-in (19-mm) mixture. The data on the shear-strength curves tend to be scattered. As the mixture becomes compacted, usually the shear strength varies. Although the trend is for the shear strength to decrease with the number of revolutions, the shear strength values are dependent on the particle orientation, which apparently causes appreciable fluctuation during compaction. An example of compaction curves (air voids versus revolutions) for the same mixtures is shown in Figure 12. These data tend to provide smooth curves, which indicate that the air voids change at a reasonably consistent rate with increasing revolutions.

The average shear strength for each mixture is shown in Table 3. An analysis of variance test was run on these shear strengths. Neither aggregate type, aggregate size, nor n value was found to be significant at a probability level of 0.95 percent. At least part of the reason for the lack of a statistically significant difference is the magnitude of the within-test variability of the shear strengths.

The GSI value measures the increase of the gyratory angle during compaction. A GSI value of about 1.1 or less indicates that the mixture is stable; whereas, a value above 1.1 indicates a tendency toward plasticity. The averages in Table 3 for which all GSI values exceed 1.1 show that the granite generally produces a more stable mixture than the quartzite. With the granite, the mixtures at an n value of

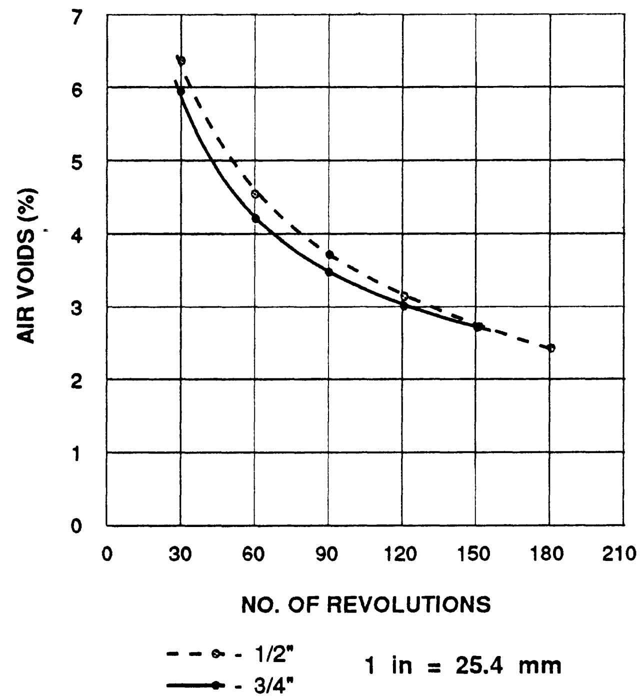


Figure 12. Air voids v. number of revolutions (quartzite, n = .4).

Table 3
RESULTS OF GYRATORY TESTING MACHINE TESTS

Quartzite

n	Maximum Size Aggregate	Shear Strength (psi)	GSI	VTM
.4	1/2 in	61	1.24	2.7
.4	3/4 in	79	1.33	2.1
.53	1/2 in	66	1.22	2.6
.53	3/4 in	57	1.20	2.4
.6	1/2 in	88	1.18	1.6
.6	3/4 in	69	1.12	1.7

Granite

n	Maximum Size Aggregate	Shear Strength (psi)	GSI	VTM
.4	1/2 in	59	1.12	2.9
.4	3/4 in	73	1.14	2.9
.53	1/2 in	58	1.09	1.5
.53	3/4 in	72	1.02	1.7
.6	1/2 in	76	1.02	1.5
.6	3/4 in	74	0.99	1.5

1 in = 25.4 mm.
1 psi = 6.89 RPa.

0.4 appear to be on the borderline of the plasticity limit, although the absolute values of shear strength indicate a reasonably strong mixture (for 120 psi [827 kPa] roller pressure, an adequate shear strength is 38.6 psi [266 kPa]).[1] The high percentage of material passing the No. 200 (75 μm) sieve (11 and 14 percent) most likely accounts for the borderline GSI values.

The VTM data do not differentiate between aggregate type or maximum size aggregate. For maximum size aggregates, there are few consistent differences within the quartzite. The VTM values for the $n = 0.6$ mixtures compact to a slightly lower air void level than do mixtures with $n = 0.4$ or 0.53. With the granite, both the $n = 0.53$ and 0.6 compact to a lower air void level than do the $n = 0.4$ mixtures. The VTM values indicate that the compactive effort of the gyratory compactor is higher than that used by the Marshall 75-blow design procedure. A VTM value of 4.0 percent was used to design the mixtures (using Marshall compaction), and the GTM produced mixtures with appreciably fewer voids (this confirms results found in a current Virginia Transportation Research Council study).[13]

Creep Test

The creep tests were run with a stress of 30 psi (210 kPa) and at a temperature of 104°F (40°C). The average creep modulus at 10 seconds and 60 minutes and the percentage of unrecovered strain (vertical) for each mixture is shown in Table 4. The results are very similar to the GTM results in that aggregate type appears to have more influence than either the n value or maximum size of the aggregate. Only one maximum size aggregate appears to be significantly different at a given n value: the granite at an $n = 0.53$ where the creep modulus values for the 3/4-in (19-mm) aggregate are higher than those for the 1/2-in (13-mm) aggregate. These two mixtures are plotted according to the method used in the AAMAS study (see Figure 13). Using this type of plot as a method of comparison, the quartzite mixtures typically plot in the area of high potential rutting at 10-second loading time but cross the moderate rutting and move into low rutting potential at the 60-minute loading time. Also, the granite mixtures tend to have higher creep modulus values (Table 4) and plot in the moderate rutting potential at the 10-second loading time and move into the low potential rutting zone at the 60-minute loading time. More importantly, the flatness of the creep curves suggest that most of the deformations are elastic and/or plastic (i.e., time independent) and not viscoelastic or viscoplastic (i.e., time dependent). The flat slopes of these curves indicate that none of these mixtures tends to creep; thus, none should have a tendency to rut.

The values of the percentage of unrecovered strain are low for all mixtures, which is consistent with the results of the creep modulus lists. The unrecovered strain values measured on these specimens suggest that most of the applied strains are elastic for both aggregate types and gradations. However, there are no consistent significant differences between either maximum size aggregates or n values. There is a tendency for the granite to have a higher percentage of unrecovered strain than the quartzite.

Table 4
CREEP RESULTS

Quartzite

n	Maximum Size Aggregate	Creep Modulus (psi) at 10 sec	Creep Modulus (psi) at 60 min	Percentage Unrecovered Strain
.4	1/2 in	14,600	10,000	18.6
.4	3/4 in	14,000	9,500	23.6
.53	1/2 in	12,500	9,000	23.0
.53	3/4 in	14,000	10,300	21.6
.6	1/2 in	12,600	9,500	20.1
.6	3/4 in	12,700	9,600	24.0

Granite

n	Maximum Size Aggregate	Creep Modulus (psi) at 10 sec	Creep Modulus (psi) at 60 min	Percentage Unrecovered Strain
.4	1/2 in	20,000	13,600	25.3
.4	3/4 in	19,800	13,200	30.5
.53	1/2 in	17,200	12,300	22.4
.53	3/4 in	20,200	14,700	25.1
.6	1/2 in	15,500	11,100	33.1
.6	3/4 in	16,700	12,000	29.3

1 in = 25.4 mm.
1 psi = 6.89 RPa.

Resilient Modulus

The resilient modulus tests were run with the Retsina Mark II device at 104°F (40°C). The averages, standard deviations, and coefficient of variations (CV %) are shown in Table 5. These test results are very similar to those of the gyratory and creep tests, i.e., there is more difference resulting from aggregate type than from either n value or maximum size aggregate. For n = 0.53, the 3/4-in (19-mm) maximum size aggregate has a higher resilient modulus than does the 1/2-in (13-mm) maximum size aggregate for both granite and quartzite. The granite has consistently higher resilient modulus values than has the quartzite. Although the granite has higher modulus values, the modulus values for the quartzite are considered acceptable.

Indirect Tensile Strength and Failure Strain

Another strength test, also taken from AAMAS, is the indirect tensile strength test with an accompanying strain-at-failure (horizontal) result. These tests were performed at 77°F (25°C) using a deformation rate of 2 in (5 cm) per minute. The averages and standard deviations are shown in Table 6. The strain-at-failure values measured on both mixes are extremely high compared to other

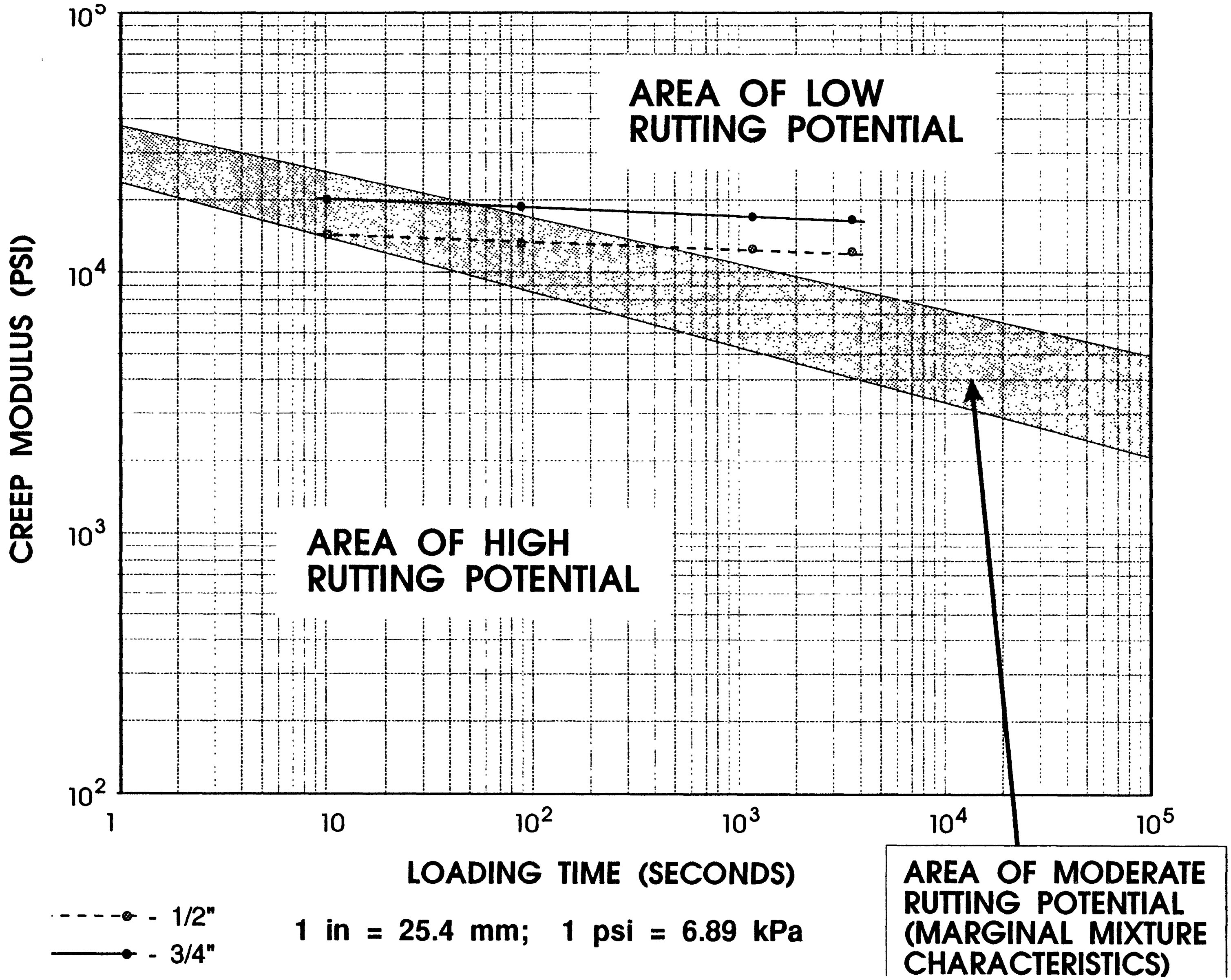


Figure 13. Creep modulus v. time (granite n = .53).

Table 5
RESILIENT MODULUS RESULTS

Quartzite

n	Maximum Size Aggregate	Resilient Modulus (psi) $\bar{X}$	S	CV (%)
.4	1/2 in	52,343	10,290	19.7
.4	3/4 in	54,129	6758	12.5
.53	1/2 in	47,600	219	0.5
.53	3/4 in	73,683	4,915	6.7
.6	1/2 in	49,653	5,099	10.3
.6	3/4 in	46,936	3,653	7.8

Granite

n	Maximum Size Aggregate	Resilient Modulus (psi) $\bar{X}$	S	CV (%)
.4	1/2 in	116,473	24,135	20.7
.4	3/4 in	87,972	2,465	1.9
.53	1/2 in	73,602	15,611	21.2
.53	3/4 in	119,448	11,029	9.2
.6	1/2 in	100,815	8,635	8.6
.6	3/4 in	101,551	27,016	26.6

1 in = 25.4 mm.
1 psi = 6.89 RPa.

Table 6
INDIRECT TENSILE STRENGTH AND STRAIN AT FAILURE

Quartzite

n	Maximum Size Aggregate	Indirect Tensile Strength (psi)		Failure Strain (mil/in)	
		$\bar{X}$	S	$\bar{X}$	S
.4	1/2 in	85.8	7.7	105.6	5.2
.4	3/4 in	80.0	8.4	105.6	8.5
.53	1/2 in	77.5	7.0	99.1	3.7
.53	3/4 in	87.8	9.6	102.3	2.5
.6	1/2 in	78.5	8.8	100.6	7.5
.6	3/4 in	69.8	3.2	99.8	6.5

Granite

n	Maximum Size Aggregate	Indirect Tensile Strength (psi)		Failure Strain (mil/in)	
		$\bar{X}$	S	$\bar{X}$	S
.4	1/2 in	89.8	6.5	105.6	5.1
.4	3/4 in	119.2	13.7	90.0	9.0
.53	1/2 in	55.1	3.9	92.4	20.5
.53	3/4 in	53.9	3.1	96.8	8.2
.6	1/2 in	55.7	4.3	98.6	9.2
.6	3/4 in	52.8	10.5	80.0	15.2

1 in = 25.4 mm.
1 psi = 6.89 RPa.

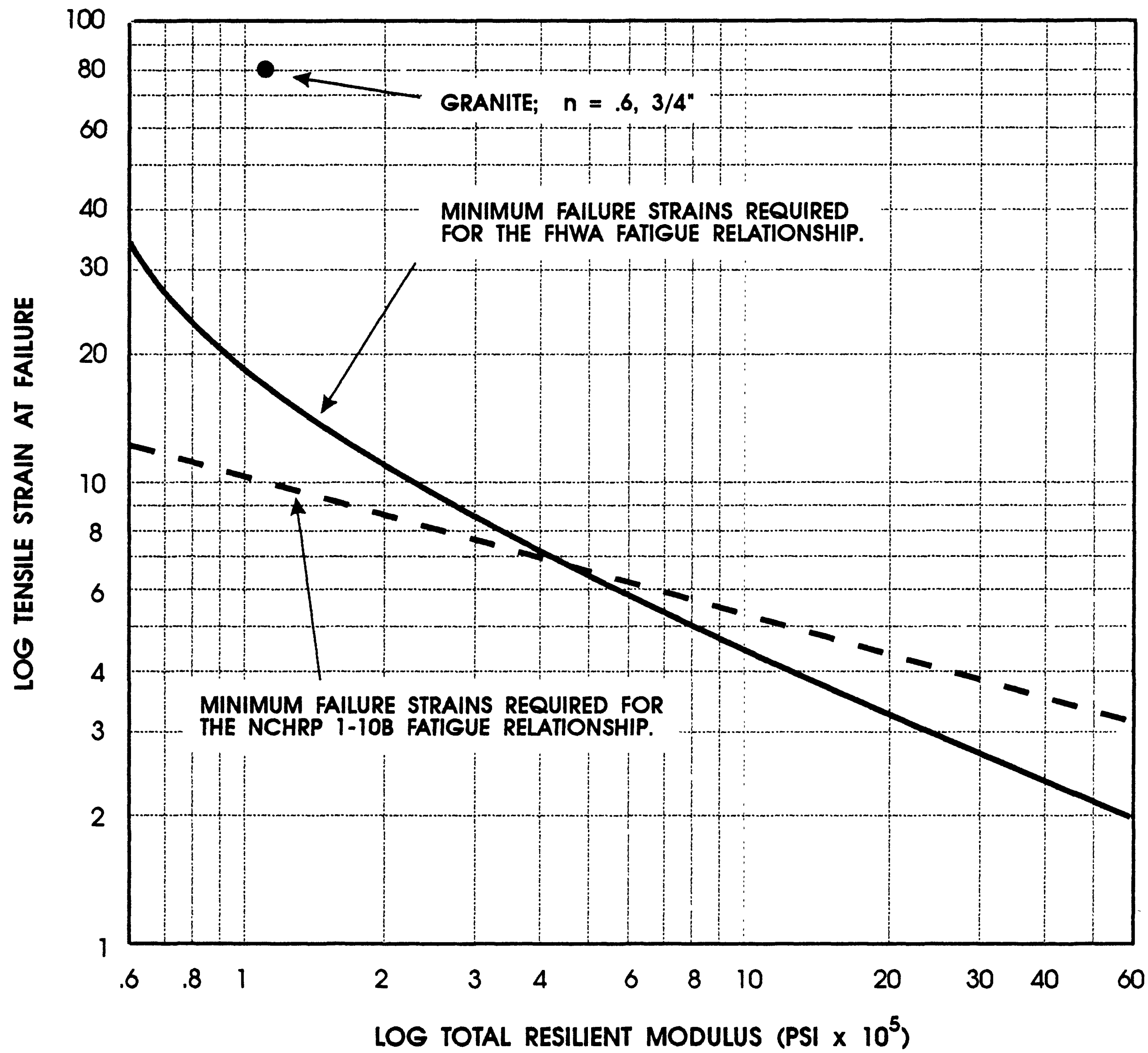
values reported in the literature.[12] The reason for these high values is unknown. These results are somewhat at odds with the earlier results in that the tendency is for the quartzite to have higher values than the granite. This is particularly true for the $n = 0.53$ and $n = 0.6$ mixtures.

The analysis used in the AAMAS study as an estimate of resistance to fatigue was used here. This analysis uses a combination of resilient modulus and tensile strain at failure (Figure 14). It should be pointed out, that the resilient modulus shown in Figure 14 and the modulus values reported in Table 5 are not the same values. The modulus values reported in Table 5 were measured with the Retsina Mark II device, as noted above. Use of this device determines a modulus value from elastic, plastic, viscoelastic, and viscoplastic deformations. However, the total resilient modulus value to be used with Figure 14 is based on only the elastic and viscoelastic deformations. Typically, the modulus values determined with the Retsina device will be much smaller than the total resilient modulus values measured with other devices in accordance with ASTM D 4123. This difference, as related to this study, is considered unimportant, because only relative comparisons are being made. Relating the lowest tensile strain at failure (Table 6) (granite; $n = 0.6$; 3/4-in [19-mm] maximum size) to resilient modulus (Table 5) indicates that this mixture would not be susceptible to fatigue failure. Although the quartzite mixtures had lower resilient modulus values than the granite mixtures, the tensile strains at failure exceeded the failure criteria and thus also would not be subject to fatigue distress.

The indirect tensile strength results are fairly consistent for the quartzite. However, with the granite, indirect tensile strengths with $n = 0.4$ are appreciably higher than those with $n = 0.53$ and 0.6. It is surmised that the higher strengths are influenced by the granite fines (which under a microscope appear more elongated than the quartzite fines) and the sand added to the mixture to increase the VMA.

Implication of Results

The series of tests run on these mixtures indicate that all the mixtures chosen should perform very well. They are all resistant to permanent deformation and fatigue, which are two of the most important kinds of distress. With the aggregates and mixtures tested in this study, the aggregate type is a larger source of difference than either the maximum size of the aggregate or the gradation used. Although the 3/4-in (19-mm) maximum size aggregate tended to be slightly stronger than the 1/2-in (13-mm) maximum size (especially for the mixtures using $n = 0.53$), the differences were erratic and of insufficient magnitude to draw any statistically based conclusion. Likewise, the results attributable to the n value were not sufficiently different to indicate an overall advantage or disadvantage of a fine versus a coarse mixture. The results tend to indicate that adequate mixture strength can be developed in several widely differing gradations. However, the $n = .4$ gradations with large amounts of – No. 200 material, which resulted in very thin film thicknesses, would produce mixtures with poor durability characteristics.



1 in = 25.4 mm; 1 psi = 6.89 kPa; 1 mil/in = 1000 m/m

Figure 14. Relationship between indirect tensile strain and resilient modulus as an indication of resistance to fatigue.

The reason for the lack of measured differences was sought. The creep specimens were sawed lengthwise to produce a cross-section that would allow a visual observation of the aggregate distribution in the various mixtures. These are shown in Figures 15 through 26. It is apparent that a great deal of aggregate-to-aggregate contact is present in all the mixtures. This is attributed to the way each gradation was developed. Irrespective of the value used for n, the fact that an n was used means that there is a systematic relationship in the amount passing each successive sieve, which in turn means that the amount passing the No. 8 sieve, for example, is proper to "fit into" the voids in the percentage passing the No. 4 sieve and so forth. Even though it is naive to think that a contractor could put a mixture together using this precision, it does seem possible and even advantageous to develop gradation bands based on this approach. In fact, since the ASTM D-3515 was used to produce the $n = 0.53$ gradations, it appears that a concept similar to this may have been used to develop the ASTM band.

In Figures 15 through 20, the quartzite mixtures show distinctively the absorptive nature of the aggregate. Several particles of the coarse aggregate have the discoloration caused by the asphalt being absorbed into the aggregate. This same phenomenon is absent in the low absorptive granite (Figures 21 through 26). Figure 21 shows what often happens in mixtures with relatively small amounts of coarser (+ 3/8 in [10 mm]) aggregate: a nonhomogeneous sample can be produced.

CONCLUSIONS

1. Because the quartzite had high absorption values, the volumetric properties varied widely depending on whether the effective or bulk specific gravity of the aggregate was used. This did not occur with the granite, which had relatively low absorption values. There were no apparent differences in the sensitivity of the coarse and fine mixtures to changes in asphalt content.

2. The estimated average film thickness of mixtures with an n value of 0.4 were so low that these mixtures are undesirable from the standpoint of durability.

3. The GTM did not indicate significant differences in shear strength or GSI for mixtures with different maximum size aggregate. VTMs obtained by the GTM mixture were lower than those obtained by the design procedure. This indicates that the GTM at 150 revolutions or more applies a higher compactive effort than a 75-blow Marshall compactive effort. The practical significance of this is that the compactive effort for the GTM might be comparable to traffic compaction on extremely high ESAL pavements

4. The creep tests indicate that all of the mixtures tested have a tendency to resist permanent deformation. There is also a tendency for the granite to have a higher percentage of unrecovered strain than the quartzite, which may make it more susceptible to permanent deformation under extremely adverse traffic conditions.

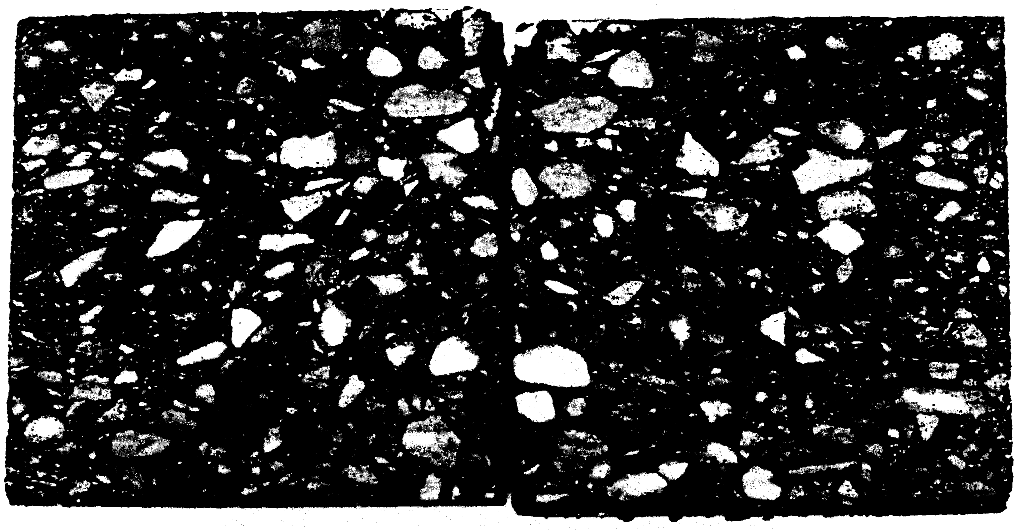


Figure 15. Quartzite (n = .4), 1/2 in (13 mm) maximum size aggregate.

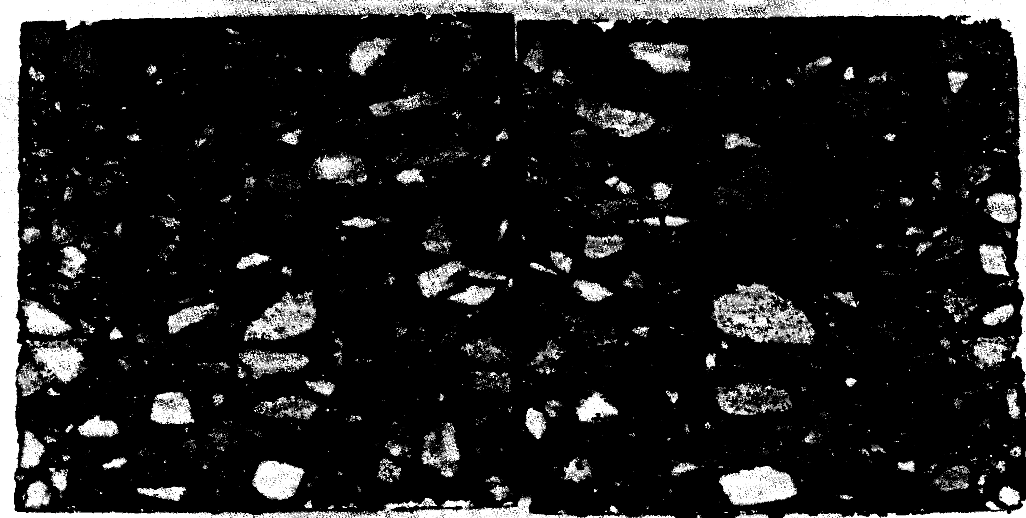


Figure 16. Quartzite (n = .4), 3/4 in (19 mm) maximum size aggregate.

Figure 17. Quartzite (n = .53), 1/2 in (13 mm) maximum size aggregate.

Figure 18. Quartzite (n = .53), 3/4 in (19 mm) maximum size aggregate.

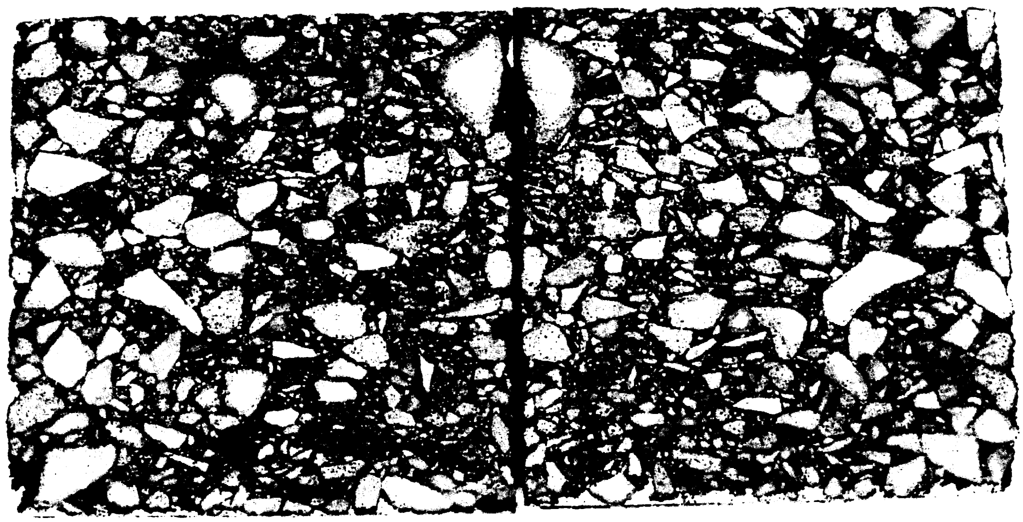


Figure 19. Quartzite (n = .6), 1/2 in (13 mm) maximum size aggregate.

Figure 20. Quartzite (n = .6), 3/4 in (19 mm) maximum size aggregate.

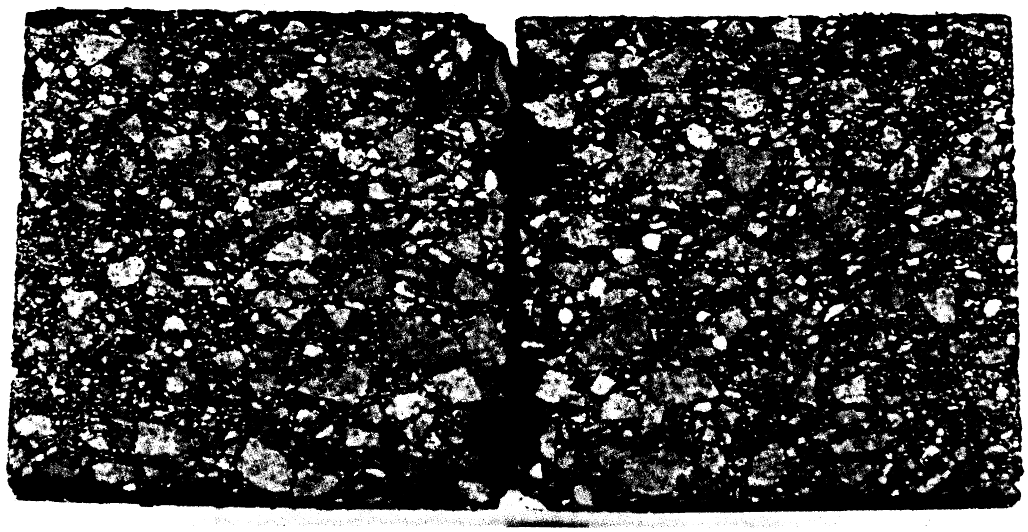


Figure 21. Granite (n = .4), 1/2 in (13 mm) maximum size aggregate.

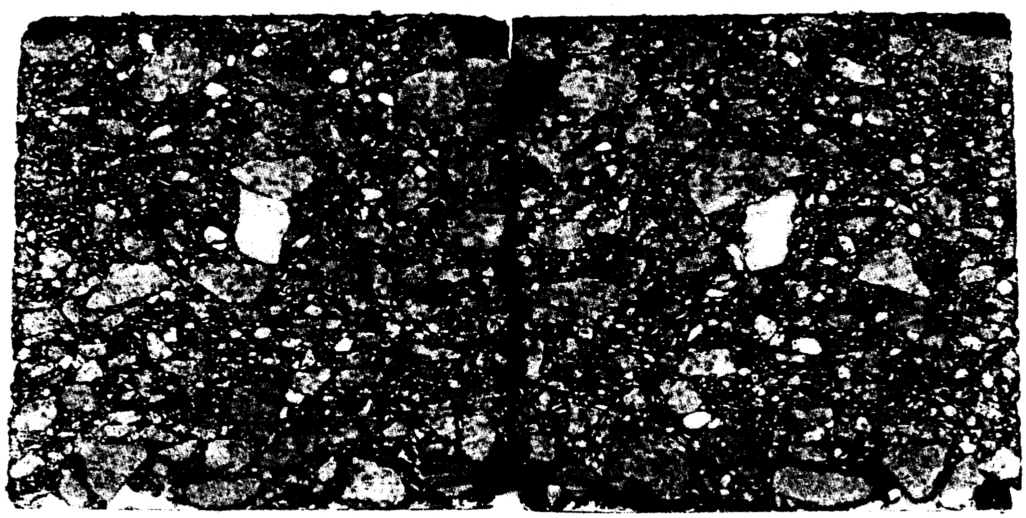


Figure 22. Granite (n = .4), 3/4 in (19 mm) maximum size aggregate.

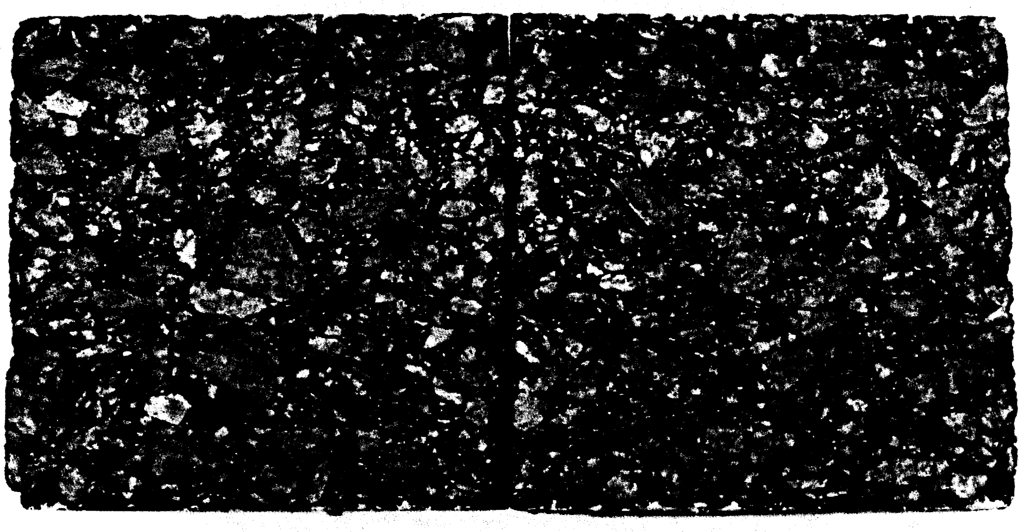


Figure 23. Granite (n = .53), 1/2 in (13 mm) maximum size aggregate.

Figure 24. Granite (n = .53), 3/4 in (19 mm) maximum size aggregate.

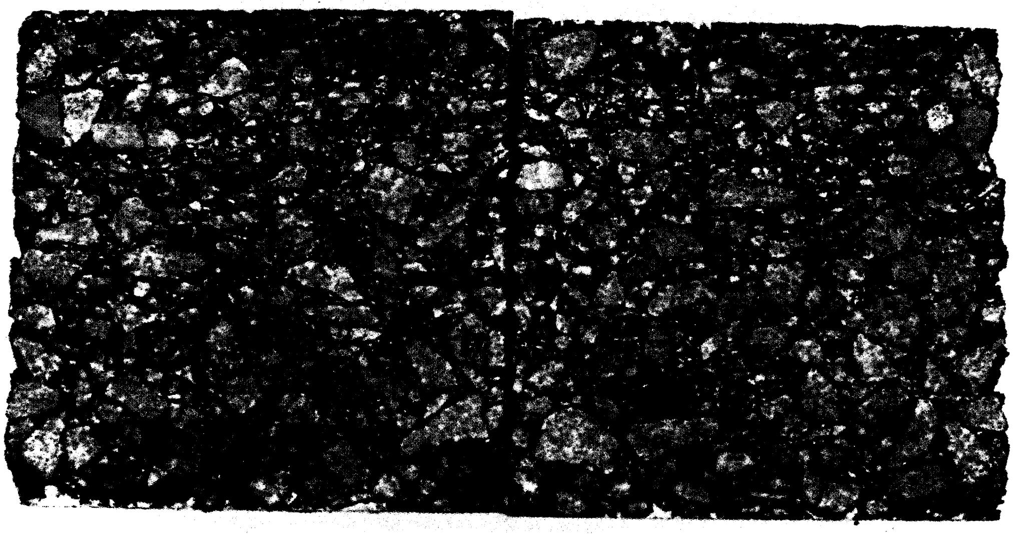


Figure 25. Granite (n = .6), 1/2 in (13 mm) maximum size aggregate.

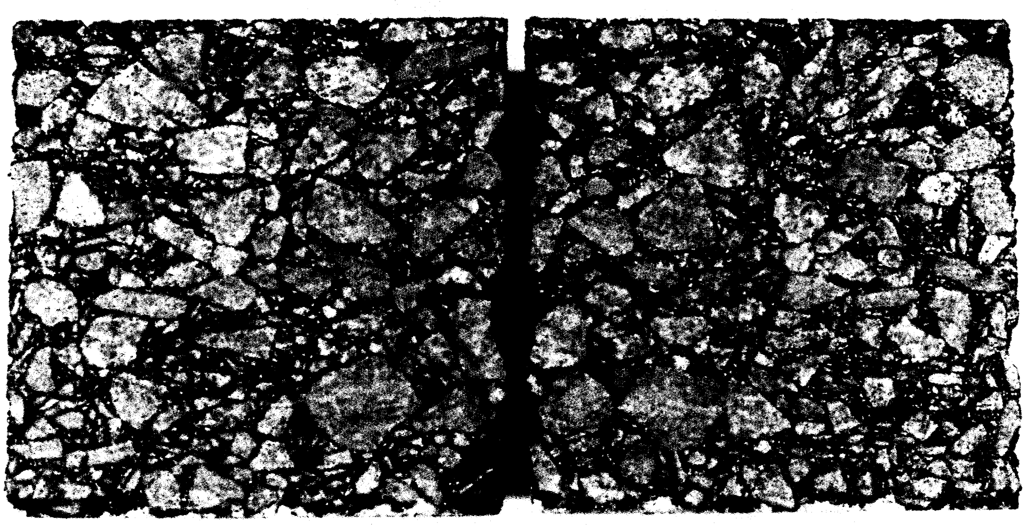


Figure 26. Granite (n = .6), 3/4 in (19 mm) maximum size aggregate.

5. Resilient modulus values were significantly higher for the granite mixtures than for quartzite.
6. The results for the resilient modulus and tensile-strain-at-failure tests, indicate that all mixtures tested would be resistant to fatigue.
7. The only mixtures for which aggregate size was a consistently significant factor were those with $n = 0.53$. For these mixtures, the 3/4-in (19-mm) aggregate mixtures were stiffer than the 1/2-in (13-mm) aggregate mixtures.
8. With all test methods, the aggregate type appears to be more significant with respect to differences in mixture characteristics than either aggregate maximum size or n value.
9. Advantages or disadvantages in strength of a fine mixture compared to a coarse mixture were not revealed with the aggregates and gradations used in this study.

RECOMMENDATIONS

When VDOT decides the present gradation design ranges need improving, the use of $n = 0.4$ and $n = 0.6$ as slopes on the percentage-passing-versus-log-sieve-size plot will provide a range of gradations that provide a systematic relationship in the amount passing each successive sieve with a maximum percentage passing the No. 200 sieve being held to 6 percent. At the same time, thought needs to be given to opening the range of percentages passing the largest sieve specified; for example, 97 to 100 percent passing the 3/4 in (19 mm) sieve for a 3/4 in (19 mm) maximum size aggregate does not allow much flexibility, but requires the percentage retained on the 3/4 in (19 mm) to be compatible with widely differing percentages passing the next smallest sieve.

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