THE DESIGN OF DENSE GRADED ASPHALT CONCRETE PAVEMENTS

BY

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ABSTRACT

The Asphalt Institute method of Marshall design is recommended in principle for dense graded asphalt concrete paving mixtures. The asphalt content of a paving mixture should be separated into the binder on the outside of the aggregate and the portion that is absorbed into the capillaries of the aggregate particles. The aggregate's ASTM bulk specific gravity should be used for paving mixture design, which should also be based on percent air voids and percent minimum VMA. When Asphalt Institute design is enforced, there is little or no advantage in using a high percentage of mineral filler. Ontario aggregate grading bands for surface and binder courses are endorsed.

PVN, a measure of temperature susceptibility, appears to provide a fingerprint that remains with a paving asphalt throughout the life of the pavement of which it is a part. A chart which divides paving asphalts into three groups of temperature susceptibility - A (low), B (medium) and C (high) - should be an essential part of every paving asphalt specification. To substantially increase a pavement's viscosity at 135 C for any penetration at 25 C, a small percentage of a suitable polymer can be incorporated.

A method for designing recycled paving mixtures in climates with and without frost is described.

I. INTRODUCTION

This paper will present subject matter under the following headings:

1. Void properties: their evaluation and selection
2. Aggregate gradation
3. Influence of mineral filler
4. Paving asphalt selection:
   (a) for regions with frost
   (b) for regions without frost
5. Pavement rutting
6. Design of recycled paving mixtures:
   (a) for regions with frost
   (b) for regions without frost

Different design procedures for dense graded asphalt concrete are in use around the world. However, the writer uses and would recommend in principle the design method of The Asphalt Institute for the reasons given in this paper.

II. VOID PROPERTIES: THEIR EVALUATION AND SELECTION

1. Voids Properties

Figure 1 is a picture of an asphalt pavement that was taken many years ago in a large South American city. Five years before this picture was taken, this city had paved 20 miles of its city streets. When I was there five years later, all of its streets looked like those in Figure 1. One could not take two steps in any direction without crossing a crack or even many cracks. Along the street car rails, as shown in the picture, the asphalt pavement had been reduced to black rubble. I was told that the asphalt content of the paving mixture as laid, had been 6.0 percent.

Some might suggest that the cracks shown in the picture are alligator cracks that are always associated with a weak foundation. However, the soil at this location was sandy, the city was in a very dry region, and the pavement cracks in the picture are shrinkage cracks.

When asked where the aggregate for the paving mixture had come from, I was taken to the banks of a dry river on the edge of the city. I filled a couple of bags with this aggregate as samples. Later, I had an opportunity to determine the aggregates specific gravity. When I placed a sample of the oven-dried aggregate in water, about half the particles would float. The aggregate was a hard volcanic pumice. Much of the bitumen that had been mixed with aggregate had been gradually absorbed into the internal capillary porosity of the individual particles of aggregate. Only a thin very brittle coating was left on the outside of each aggregate particle to serve as a binder.

This experience taught me a lesson about asphalt paving mixture design that I have never forgotten, namely: the total bitumen content of every asphalt paving mixture is divided into two parts. First there is the portion that remains as a coating on the outside of each individual aggregate particle, the "effective bitumen content" that provides a pavement with its bituminous binding material. Second, there is the portion that is absorbed into the internal capillary porosity of each individual aggregate particle, and contributes litte or nothing as a bituminous binder to the paving mixture.

Figure 2(a), which represents the junction of three bitumen coated aggregate particles in a compacted paving mixture, illustrates the voids properties of a typical asphalt paving mixture, in which a part of the total bitumen content of the paving mixture is almost always lost by absorption into the internal capillary porosity of the individual aggregate particles. The exaggerated wedges in Figure 2(a) represent internal capillary pore space within each individual particle of aggregate. Figure 2(a) demonstrates that these internal capillary pore spaces are partly filled with absorbed bitumen and partly with air. It is well known that the internal capillary pore space within the individual aggregate particles will absorb less bitumen than water.

The implications of Figure 2(a) can be more easily understood by referring to Figure 2(b), which will be used from now on. Figure 2(b) also represents the juncture of three bituminous coated aggregate particles in a compacted paving mixture.

The outermost circles in Figure 2(b) represent the volume of aggregate in a paving mixture that is given by the aggregate's ASTM bulk specific gravity. The innermost circles represent the volume of the aggregate that is given by the aggregate's ASTM apparent specific gravity. The intermediate circles represent the volume of the aggregate that is given by the aggregate's virtual specific gravity (sometimes erroneously referred to as the aggregate's "effective" specific gravity).

The black partial circles in Figure 2(b) represent he portion of the total bitumen in a compacted paving mixture that forms the coating on the outsides of the aggregate particles, and which functions as the bituminous binder in an asphalt pavement. The portion of the total bitumen that is absorbed into the internal capillary porosity of each individual aggregate particle in a paving mixture is represented by the hatched area between the outermost and intermediate circles. The absorbed bitumen has little or no influence as a binder on paving mixture properties, but may contribute to anti-stripping.

The area between the intermediate and the innermost circles in Figure 2(b), represents the portion of the internal capillary porosity within each aggregate particle that is not filled with absorbed bitumen, and in a well-prepared hot-mix paving mixture is filled with air.

Figure 2(b) illustrates all the information that a specification writer or a paving engineer needs for the intelligent design of a dense graded bituminous concrete paving mixture. It has been available since the publication of my ASTM paper in 1959 in ASTM's Special Technical Publication No. 252.

2. Definitions for Percent Air Voids and Percent VMA

From Figure 2(b), it is clear that percent air voids and percent VMA (voids in the mineral aggregate in a compacted paving mixture) can each be defined in any one of three quite different ways. The three possible definitions for percent air voids are as follows:

(a) The total volume of the void spaces between the bitumen coated aggregate particles that are scattered all through a sample of compacted paving mixture, expressed as percent of the bulk volume of the sample of paving mixture.

(b) The total volume of the void spaces between the bitumen coated aggregate particles that are scattered all through a sample of compacted, dense graded paving mixture, plus the portion of the internal capillary porosity within each individual aggregate particle in the sample that is not filled with absorbed bitumen, which in Figure 2(b) is represented by the area between the intermediate and innermost circles, expressed as percent of the bulk volume of the sample of compacted paving mixture.

(c) The total volume of the portion of void space within the internal capillary porosity of each individual aggregate particle in a sample of compacted paving mixture, that is not filled with absorbed bitumen, which in Figure 2(b) is represented by the area between the intermediate and innermost circles, expressed as percent of the bulk volume of the compacted paving mixture sample.

It is also clear from Figure 2(b), that three different definitions are possible for percent VMA, which is the bulk volume of a sample of compacted paving mixture minus the volume of the aggregate in the sample, expressed as percent of the bulk volume of the sample.

(a) VMA is the total volume of intergranular void space in a sample of compacted paving mixture, that is determined as the difference in volume between the bulk volume of the sample and the volume of the aggregate in the sample that has been calculated from the aggregate's ASTM bulk specific gravity (represented by the outermost circles in Figure 3(b), and expressed as a percent of the bulk volume of the sample of compacted paving mixture.

(b) VMA is the total volume of void space in a sample of compacted paving mixture, that is determined as the difference in volume between the bulk volume of the sample, and the volume of the aggregate in the sample that is calculated by the aggregate's virtual (effective) specific gravity (represented by the intermediate circles in Figure 2(b)), and expressed as percent of the bulk volume of the compacted sample of paving mixture.

(c) VMA is the total volume of void space in a sample of compacted paving mixture, that is determined as the difference in volume between the bulk volume of the sample, and the volume of the aggregate in the sample that is calculated from the aggregate's ASTM apparent specific gravity (represented by the innermost circles of Figure 2(b), and expressed as percent of the bulk volume of the sample of compacted paving mixture.

3. Selection of Definitions for Percent Air Voids and Percent VMA

All three of the above definitions for percent VMA, and probably at least two of the above definitions for percent air voids are currently specified and employed by different agencies on this continent. This leads to the question, "Which of these three definitions for percent air voids and percent VMA should be selected?". The choice is very easy if we state simply that the definitions chosen for percent air voids, and for percent VMA must agree with good pavement performance on the road.

(a) Definition for Percent Air Voids

On this basis, the definitions for percent air voids must be the first one listed above, namely, the total volume of the small pockets of air between the bitumen coated aggregate particles in a sample of compacted paving mixture, expressed as percent of the bulk volume of the sample. Experience has shown that it is when these air voids approach one percent that a pavement begins to flush or bleed, or to show other signs of pavement instability.

The second possible definition for percent air voids above, "Void spaces between the bitumen coated aggregate particles in a sample of compacted paving mixture plus the portion of the internal capillary porosity within each aggregate particle in the sample that is not filled with absorbed bitumen", is untenable because the void space between the coated aggregate particles could be approaching one percent, with the pavement actually flushing or bleeding, or otherwise unstable for this reason, but an air voids measurement could still show a substantial percent in extreme cases, (possibly even approaching the minimum air voids requirement of 3 percent), because it would represent the portion of the internal capillary porosity of the individual aggregate particles that was not filled with absorbed bitumen. This definition therefore, would not meet the requirement that it must agree with pavement performance on the road; since at from 2 to 3 percent air voids, as percent air voids is defined in (a) above, the pavement would not be flushing or bleeding.

The third possible definition for percent air voids given above, would appear to have no relationship with good pavement performance on the road, and is not considered to be a practical definition for this reason.

(b) Definition for Percent VMA

Like an acceptable definition for percent air voids, an acceptable definition for percent VMA, should also require a very close relationship between the definition for percent VMA and good pavement performance on the road.

On this basis, only the first definition given earlier qualifies, namely, "VMA is the total volume of intergranular void space in a sample of compacted paving mixture, that is determined as the difference in volume between the bulk volume of the sample and the volume of the aggregate in the sample which is calculated from the aggregate's ASTM bulk specific gravity (represented by the outermost circles in Figure 2(b), and expressed as percent of the bulk volume of the sample of compacted paving mixture". Only this definition makes it necessary to divide the total bitumen content of a paving mixture into the portion that remains as a coating on the outsides of the aggregate particles, and the portion that is absorbed into the internal capillary porosity within each individual aggregate particle, where it has disappeared with respect to any contribution it might make to good pavement performance, as demonstrated by Figure 1 and Figure 2(b).

If VMA is defined as the difference in volume, between the bulk volume of a sample of compacted paving mixture and the volume of the aggregate in the sample calculated from the aggregate's virtual (effective) specific gravity (definition (b) above), or as calculated from the aggregate's ASTM apparent specific gravity (definition (c) above), expressed as percent of the bulk volume of the sample, the assumption is being made that the total bitumen content of the paving mixture exists as a coating on the outsides of the aggregate particles, and that bitumen absorption into the internal capillary porosity of each individual aggregate particle does not occur and is zero, Figure 2(b). It is clearly demonstrated by Figure 1 that neither of these two definitions for VMA is related to pavement performance, and for this reason, both definitions (b) and (c) above are invalid.

It should be added that both acceptable definitions for percent air voids and percent VMA given above, are also the definitions employed by The Asphalt Institute.

4. Voids Properties of an Actual Compacted Paving Mixture

Figure 3 illustrates the acceptable definitions for percent air voids and for percent VMA, as given above, for an actual sample of compacted paving mixture, for example, a Marshall briquette.

(a) The total bitumen content is divided into the "effective" bitumen content, which is the portion of the total bitumen content that exists as a coating on the outsides of the aggregate particles, and the portion of the total bitumen content that is lost by absorption into the internal capillary porosity within each of the aggregate particles, as indicated by Figure 1 and Figure 2(b).

(b) The percent air voids consists of the total volume of the small pockets of air between the coated aggregate particles, that are distributed throughout a sample of compacted paving mixture.

(c) The volume of aggregate is shown in terms of the aggregates ASTM bulk specific gravity. Figure 3 clearly demonstrates that a portion of the total asphalt content is absorbed into the internal capillary porosity of each individual aggregate particle.

(d) Percent VMA is shown to consist of the total volume of the sample of compacted paving mixture minus the volume of the aggregate in the sample that is given by the aggregate's ASTM bulk specific gravity. The same value for percent VMA is obtained by adding together the percent by volume of the air voids plus the percent by volume of the effective bitumen content.

5. Influence of ASTM Bulk, ASTM Apparent and Virtual (Effective) Specific Gravities of the Aggregate on the Voids Values for a Paving Mixture

Table 1 illustrates the differences in voids values for a compacted dense graded paving mixture made with crushed gravel aggregate that we have been using in our laboratory for some time for research purposes. This table demonstrates the differences in voids values that result from using the aggregate's ASTM bulk specific gravity, ASTM apparent specific gravity and the virtual (effective) specific gravity of the aggregates.

Table 1, demonstrates that the percent air voids values are the same, whether the ASTM bulk specific gravity, 2.675, or the virtual specific gravity of the aggregate, 2.734, are used for paving mixture design. However, when the aggregate's ASTM apparent specific gravity, 2.794, is employed for this purpose, the percent air voids is higher by 5.2 - 3.4 = 1.8 percent than when the paving mixture design is based on the aggregate's ASTM bulk specific gravity, 2.675. This air voids difference of 1.8 percent consists of the portion of the volume within each individual aggregate particle that is not filled with absorbed bitumen, as shown by Figures 2(a) and 2(b). It has no effect on pavement performance, and therefore cannot be considered to be part of the normal air voids.

It is clearly shown by Table 1, that because of the partial loss of the total bitumen content of the paving mixture by absorption into the internal capillary porosity of the individual aggregate particles, Figure 2(b), the VMA value, 17.6 percent, given when the virtual (effective) specific gravity of the aggregate, 2.734, is employed for paving mixture design, it is 17.5 - 15.6 = 1.9 percent higher than the VMA value of 15.6 percent provided when the ASTM bulk specific gravity, 2.675, is utilized for paving mixture design with the same aggregate.

Also with respect to percent VMA, Table 1 indicates that for this particular paving mixture, which is not untypical of paving mixtures in Canada, when the aggregate's ASTM apparent specific gravity, 2.794, is employed for paving mixture design, the corresponding VMA value is 19.2 - 15.6 = 3.6 percent higher than when paving mixture design is based on the same aggregate's ASTM bulk specific gravity, 2.675. This VMA difference of 3.6 percent represents a very large difference in VMA values. It also represents the total volume of the capillary porosity within the individual aggregate particles in the paving mixture, 3.6 percent.

Because VMA has been calculated in the normal way (total volume of compacted sample of paving mixture minus volume of aggregate in the sample based on the ASTM apparent specific gravity), the value for percent voids filled with bitumen opposite the ASTM apparent specific gravity, 72.9 percent, in Table 1 is in error. When calculated in the normal way, the VMA value, 19.2 percent, includes the volume of the portion of the capillary porosity within each aggregate particle that is not filled with absorbed bitumen, Figure 2(b), which Table 1 shows should be the VMA shown for the ASTM apparent specific gravity, 19.2 percent minus the VMA for the virtual (effective) specific gravity 17.5 percent; = 19.2 - 17.5 = 1.7 percent. Consequently, when calculated on the basis of the ASTM apparent specific gravity for the aggregate in the paving mixture in Table 1, the actual value for percent voids filled with bitumen should be

$$\frac{\text{\% by volume of total asphalt}}{\text{corrected VMA}} = \frac{14.06}{19.2 - 1.7} = \frac{14.06}{17.5} = 80.3 \text{ Percent}$$

When calculated in the normal way therefore, on the basis of the aggregate's ASTM apparent specific gravity, there is an error of 80.5 - 72.9 = 7.6 percent in the percent voids filled with bitumen in the paving mixture of Table 1. That is, the actual or real value for percent voids filled with bitumen is 7.6 percent higher than would normally be calculated.

As previously pointed out, Table 1 shows that the air voids are 5.2 - 3.4 = 1.8 percent higher when the mix design is based on ASTM apparent specific gravity than on the aggregate's ASTM bulk specific gravity. That is, on the basis of the aggregate's ASTM apparent specific gravity, if the paving mixture were to be designed for say 3 percent air voids, at the bottom of the range of 3 to 5 percent air voids normally specified, its actual air voids value would be 3.0 - 1.8 = 1.2 percent, and the resulting pavement could be highly unstable with the development of rutting, and it could also be flushing or bleeding.

Consequently, then the aggregate's ASTM apparent specific gravity is employed for paving mixture design, the actual percent voids filled with bitumen could be substantially higher than calculated, and the percent air voids could be substantially lower. Each of these errors could result in a disastrous pavement failure.

To emphasize the point again, as demonstrated by Figures 1, 2(a) and 2(b), neither the virtual (effective) specific gravity nor the ASTM apparent specific gravity of the aggregate provides the separation of the total bitumen content into the portion that is absorbed into the internal capillary porosity within each individual aggregate particle, and the portion that remains as a coating on the outside of the aggregate particles on which pavement performance and length of pavement service life depends. Therefore, engineers would be justified in rejecting both the virtual (effective) specific gravity and the apparent specific gravity of the aggregate as a basis for paving mixture design.

6. The Role of Percent VMA, Percent Air Voids and Percent Voids filled with Bitumen, VFB, in Paving Mixture Design

Some organizations specify percent air voids and percent voids filled with bitumen as their criteria for paving mixture design. Other agencies specify percent air voids and percent VMA as their design requirements, while still others specify all three, percent air voids, percent VMA and percent voids filled with bitumen as their criteria for the design of paving mixtures.

In this connection, it should be noted that:

$$\% \text{ VFB} = \frac{\% \text{ effective bitumen}}{\% \text{ VMA}} \times 100$$
$$= \frac{\% \text{ VMA} - \% \text{ Air Voids}}{\% \text{ VMA}} \times 100$$

The relationship between percent air voids and percent voids filled with bitumen is illustrated in Figure 4. The air voids range shown is from 3 to 5 percent because it is generally accepted that this is the minimum range required for hot paving mixture production. This range is not unreasonable in view of the fact that the precision of the present test for percent air voids is roughly $\pm$ 1.0 percent.

Figure 4, as well as Figures 5 and 6 are based on an aggregate ASTM bulk specific gravity of 2.650 and a bitumen specific gravity of 1.010.

It is unusual to be able to plot the relationships between five variables on a 2-dimensional graph. Nevertheless Figures 4, 5 and 6 demonstrate that this is possible for a compacted paving mixture. The ordinate axis consists of compacted pavement density without any absorbed bitumen. The abscissa indicates the effective bitumen content (total bitumen content minus absorbed bitumen). The lines representing the variables percent air voids and percent voids filled with bitumen cross the charts roughly from upper left to lower right, while the lines representing different values for VMA travel roughly from lower left to upper right.

Figure 5 illustrates a plot of a minimum percent VMA of 15 percent and an air voids range from 3 to 5 percent. Figure 5 therefore illustrates percent VMA and percent air voids for a typical asphalt wearing course.

Figure 6 demonstrates the plot of a specification that requires 75 to 85 percent voids filled, 3 to 5 percent air voids and a minimum VMA of 15 percent. The air voids range shown in each of the three cases is from 3 to 5 percent, because it is generally agreed that this is the minimum range of percent air voids required for hot paving mixture production.

Figure 4 indicates that there is justification for the favourable comments by Vallerga, White and Rostler (2) and by Huber and Heiman (3) on the usefulness of percent voids filled with bitumen and percent air voids as the best criteria for the good performance of an asphalt pavement.

However, Figure 4 also demonstrates that percent voids filled with bitumen and percent air voids do not provide acceptable criteria for asphalt paving mixture design. For the range of bitumen ordinarily required for the optimum design of paving mixtures the percent air voids is substantially less than from 3 to 5 percent and even from 3 to 4 percent. A hot-mix plant cannot operate within a range range of less than from 3 to 5 percent air voids. Experience has also shown that the minimum air voids value to avoid pavement flushing and bleeding is 3 percent. It must also be kept in mind that the precision of the tests presently available to us for the determination of percent air voids is the average value ± 1.0 percent. That is, we do not know what the actual air voids value is within the range of plus or minus one percent from its measured value.

If the three requirements, percent air voids, percent voids filled with bitumen and percent minimum VMA are specified, Figure 6 asks the question, "which of these three criteria are to be followed?". They get into each others way.

Consequently, as a practical matter, Figure 5 illustrates the only reasonable criteria that should be specified for asphalt paving mixture design, namely, 3 to 5 percent air voids and a minimum percent VMA, that depends on the nominal maximum particle size of the aggregate.

7. The Need for Minimum VMA Requirements

It is generally agreed, that to provide asphalt pavement durability the bitumen content of a dense graded paving mixture should be as high as possible consistent with good pavement performance on the road. However, what are the minimum limits for this optimum bitumen content, and how should they vary with the nominal maximum particle size of the aggregate in the paving mixture? Figure 7 is an attempt to provide a practical answer to this problem (1). It must also be remembered that the percent VMA in a compacted paving mixture provides the only room or space between the aggregate particles that is available for the 3 to 5 percent air voids and the volume of bituminous binder that is needed for a durable pavement. If the percent VMA is too low, there is not enough of this room in the compacted paving mixture for the bitumen needed for a durable pavement plus the 3 to 5 percent air voids, and one or the other or both will be too low, with the resulting destructive effect on pavement performance.

The coordinate axes of Figure 7 are "VMA-percent" as the ordinate and nominal maximum particle size of the aggregate as the abscissa. Figure 7 indicates the boundary between acceptable minimum percent VMA values for paving mixture design (on or above the oblique boundary) versus unacceptable values for VMA (below this boundary). Figure 7 demonstrates very clearly, that when the minimum VMA values are too low (below this boundary) a paving mixture is deficient in either bituminous binder or in percent air voids or in both.

Figure 8 illustrates the VMA values obtained on pavement samples some years ago, that were cut from a pavement that was ravelling very seriously. Cores consisting of surface and base course layers were taken from eight locations on this pavement, and were analysed by Mr. J.A.A. Lefebvre of Imperial Oil's Research Department at Sarnia, Ontario. Figure 8 shows very clearly that each of the eight surface course and each of the eight base course paving mixtures were low in percent VMA, some very seriously low, and were therefore deficient in bituminous binder. This resulted in the pavement distress referred to in the note on Figure 8.

The air voids requirement for dense graded paving mixtures in most specifications is currently from 3 to 5 percent. This might seem to invite a similar spread in the minimum percent VMA requirements illustrated in Figures 7 and 8. However, as pointed out earlier, with the current ASTM testing procedures, the ASTM precision for any reported air voids value is within the range of the average value ± one percent. this means that for specifications calling for from 3 to 5 percent air voids, a paving mixture should be designed for 4 percent air voids if it is always to be within the specified range of 3 to 5 percent. As a consequence to avoid under-asphalting, the minimum VMA value specified for a paving mixture should be associated with the corresponding air voids value of four percent. That is, the minimum VMA requirements illustrated by the oblique line in Figure 7 should be specified.

Following the OPEC oil crisis in 1973, which was accompanied by a meteoric increase in the price of paving asphalt for road construction and maintenance operations, a number of engineers and contractors thought (and some still do) that they could reduce the cost of dense graded asphalt concrete pavements by substantially reducing the percent VMA requirements, which in turn would result in a significant reduction in the bitumen contents of paving mixtures. In the short term this would reduce the initial cost of asphalt pavement construction, but it lost sight of the greatly increased long term costs occasioned by early increased maintenance and greatly shortened service lives.

Ontario was one area where this reduction in percent VMA requirements for paving mixtures was tried. Mr. Daniel F. Lynch, Head Bituminous Section, Ontario Ministry of Transportation and Communications advises that wherever this change was made, it was followed by an epidemic of poor pavements that ravelled badly and cracked severely in their early service lives. Mr. Lynch states that this questionable alternative to good paving mixture design was halted quickly, and the original VMA requirements, which are patterned after those of The Asphalt Institute, and are illustrated by the oblique line in Figure 7, were restored and enforced.

In his AAPT paper for 1959, Krchma (4), a well-known authority on asphalt paving technology in the United States, after a comprehensive survey of existing information, concluded that one-half the expected service life of an asphalt pavement can be lost, if the bitumen content of a paving mixture is only one-half percent less or one-half percent more than its optimum.

8. Nominal Maximum Aggregate Particle Size

A quite important question associated with the minimum percent VMA requirements illustrated in Figure 7, is what is meant by "the nominal maximum particle size of an aggregate in a paving mixture?". In my opinion, the best practical definition for the phrase "nominal maximum particle for the aggregate in a dense graded paving mixture", has been provided by the Ontario Ministry of Transportation and Communications, namely:

> "The nominal maximum aggregate particle size for the aggregate in a dense graded asphalt paving mixture, is the next larger standard sieve size than the sieve size on which at least 10 percent of the total aggregate is retained".

For example, the coarse aggregate portion, retained on a No 4 sieve, of four aggregates could have the following gradations:

Sieve Size	Agg. 1	Agg. 2	Agg. 3	Agg. 4
	Percent Passing			
3/4 inch	100	100	100	100
5/8 inch	89	93	96	98
1/2 inch	74	82	92	95
3/8 inch	58	73	83	91
No 4 Sieve	48	54	62	88
etc.	etc.	etc.	etc.	etc.
Nominal Maximum Agg. Particle Size	3/4 inch	5/8 inch	1/2 inch	3/8 inch
Corresonding Minimum % VMA	14.0	14.5	15.0	16.0

In the absence of the above definition the average engineer would tend to assume that the nominal maximum aggregate particle size should be 3/4 inch in each case, because it is the largest particle size in each aggregate, and that the corresponding minimum percent VMA value selected should be 14.0 percent. However, with the guidance of the above definition, the "nominal maximum aggregate particle size" is given by the circled figure in each column. The successively higher VMA values along with the requirement for from 3 to 5 percent air voids, provide space for the increasingly higher minimum bitumen contents needed by these successively finer dense graded paving mixtures.

II. AGGREGATE GRADATION

The VMA of a compacted dense graded paving mixture is very largely controlled by the gradation of the aggregate. For any aggregate, a Fuller grading curve provides the densest grading and also the minimum VMA for a compacted paving mixture made with this aggregate.

The formula for a Fuller grading curve is:

$$P = \left(\frac{d}{D}\right)^{\frac{1}{2}}$$

where

P = the percent of total aggregate passing each sieve
D = the sieve size for the largest aggregate particle
d = the sieve size for any particle smaller than D

There is a whole family of Fuller grading curves, one for each maximum aggregate particle size, as illustrated by the fainter background curves in Figures 9, 10 and 11.

The effect of aggregate gradation on the VMA values of paving mixes is illustrated in Figure 9, which resulted from some research we did for the Canadian Department of National Defence some time ago (5,6). In Figure 9, the Fuller curve for the aggregate provided a wearing course paving mixture with a VMA value of only 11.5 percent. By gradually deviating from the Fuller curve by changing the aggregate gradation, the corresponding paving mixture VMA values were increased to 13.5, 15.0 and finally to 16.5 percent. The corresponding paving mixture bitumen contents associated with each of these grading curves increased from only 3.9 percent for the paving mixture with a VMA of only 11.5 percent to 6.2 percent for the paving mixture with a VMA of 16.5 percent. All of the paving mixtures in Figure 9 were designed to have an air voids content of 4 percent.

As shown by Figure 10, the Marshall stability is highest for the paving mixture with a Fuller grading associated with a VMA of 11.4 percent, and is reduced by about one-half when the VMA is increased to 16.5 percent, with the air voids value being held constant at 4 percent in all cases. Therefore, a price is paid in terms of reduced Marshall stability as the paving mixture percent VMA is gradually increased from that for a Fuller grading. Nevertheless, the specified minimum Marshall stability, minimum percent VMA and 3 to 5 percent air voids can ordinarily be easily obtained with the aggregates available.

All of these paving mixtures were made with a filler/bitumen ratio of 0.9 by weight.

The P.I. (particle index) of 11.5 shown in some of the figures refers to the effect of particle shape and particle surface texture for each aggregate sieve size, as determined by ASTM D3398. A constant P.I. value for each of the particle sizes in the aggregate in a paving mixture ensures that the influence of particle shape and particle surface texture of the various aggregate sizes present is the same and does not influence the value of the variable being investigated.

A question might be asked about the procedure employed to make an aggregate grading curve deviate from that for the corresponding grading curve. A grading curve for the crushed aggregate produced by crushing quarried rock, ordinarily follows the shape of a Fuller curve and is concave upward, Figure 11. The grading curve for a natural sand, on the other hand is usually concave downward, Figure 11. Consequently, by combining natural sand with crusher-run aggregate, or with crusher-run aggregate that has been divided into coarse and fine (screenings) fractions, depending on the blend ratios, the grading curve for the resulting aggregate can be made to deviate signficantly away from the corresponding Fuller curve, as demonstrated by Figure 9.

Occasionally, a paving mixture can be in serious trouble because its gradation is too far from its corresponding Fuller curve. This is illustrated by Mix C in Figure 12, a sample of which was sent to us some time ago. Both its percent VMA and percent air voids are much too high. Its bitumen content was 6.0 percent. The solution in this case is to move in toward the Fuller curve by changing the gradation of the aggregate. On this basis Mix C is entirely satisfactory, and at the same bitumen content of 6.0 percent.

For asphalt paving mixtures, many agencies tend to specify a relatively narrow aggregate grading band that tends to follow a Fuller curve. One wonders if this is simply a case of follow the leader, whoever that may have been, and a reluctance to try anything new. However, Ontario's grading band for wearing course is illustrated in Figure 13.

The Ontario aggregate grading band includes a bulge in its upper boundary, which enables aggregate grading curve deviations well away from the corresponding Fuller curves to be made; as illustrated in Figure 9. Ontario's grading band for base course paving mixtures is similar to Figure 13, but accommodates a larger maximum particle size. We have used the Ontario grading bands with success for many years.

III. THE INFLUENCE OF MINERAL FILLER

There has been much debate on the effect of mineral filler on asphalt paving mixtures, particularly on their Marshall stability. The writer has always handled the mineral filler problem by assuming that the mineral filler is part of the aggregate. It can be shown that if the paving mixture design adheres to The Asphalt Institute criteria, mineral filler does not have any large influence on items such as Marshall stability.

Mineral filler was included in our investigation for the Department of National Defence referred to earlier (5,6). Most mixes we tested at that time had a filler/bitumen ratio of 0.9 by weight.

For one group of tests we used a paving mixture with Fuller grading. The only change made in these paving mixtures was to allow the percent passing the No 200 sieve to change over the range from 2.0 to 3.65 to 8.0 percent. The bitumen content was varied as required, to maintain the air voids at 4.0 percent. No limits were placed on the percent VMA, or on the percent bitumen required. All mixes were compacted by 60-blows of a mechanical compactor, which was roughly equivalent to 75-blows of a Marshall hand compactor. The grading curves are shown on Figure 14.

Figure 15 illustrates the effect of these changes in mineral filler contents on the corresponding values measured for Marshall stability. As expected, the Marshall stability increased dramatically from about 1500 pounds for 2 percent mineral filler, to about 2600 pounds when the mineral filler was increased to 8.0 percent.

A more normal grading (for us), was then employed, with the VMA maintained at 15 percent and the air voids at 4 percent (as we would normally do for surface course paving mixture design), while the percent passing No 200 sieve was varied from 2.0 to 5.14 to 8.0 percent, Figure 16.

The effect of these changes in filler content and mix design on Marshall stability is shown in Figure 17. This figure indicates that no change in Marshall stability occurred with the increase in mineral filler. The explanation for this result, which initially will seem quite surprising, appears to be that the increasing bulge in the fine aggregate portion of the grading curve for the mixes containing 5.14 and 8.0 percent passing No 200, reduced the Marshall stability, Figure 10, by the same amount that it is increased by the increasing percent passing the No 200 sieve.

Consequently, when designing dense graded asphalt concrete to have constant values for percent VMA and percent air voids (Asphalt Institute design) the Marshall stability will be increased by an increase in percent passing the No. 200 sieve, only if this stability increase is greater than the loss of Marshall stability that is occasioned by the larger bulge in the fine aggregate portion of the grading curve that is required to maintain the percent VMA value specified.

IV. MARSHALL FLOW INDEX

Marshall flow index is the decrease in diameter of a Marshall test speciman at the point where "the load begins to decrease" during a Marshall test. It is measured in units of 1/100 inch or equivalent units, for example mm or 0.025 mm.

The flow index is a very important property of a paving mixture. It serves as a red flag or danger signal indicating plastic mix, for example too much bitumen, when designing a paving mixture, for controlling a hot-mix plant during construction, or when analysing a pavement sample. It does not in itself indicate what is wrong with a pavement sample, or with a sample of hot-mix taken for control, or with a mix being designed in the laboratory. However, when the flow index is higher than from 18 to 20 units of 1/100 inch, it indicates that the mix is plastic or is nearing the plastic stage and is likely to show or develop instability problems such a rutting, rippling, pushing, etc., and even flushing or bleeding.

The flow index is not ordinarily useful as an indicator of an underasphalted mix, because the high air voids usually associated with this condition, tend to maintain the flow index at a normal level of from 9 to 16 units of 1/100 inch. However, when the flow index is high, it is a signal to check one or more items such as percent air voids, percent VMA, percent bitumen content, aggregate gradation and perent passig the No 200 sieve, as possible causes. If the flow index is unusually low, the same variables require checking.

The procedure specified in ASTM D1559 for measuring the flow index is:

> "Release the flow meter......the instant the maximum load begins to decrease".

This can lead to personal error when using the flow meter to measure flow index. In our laboratory, we use a stress/strain recorder, which draws a graph of Marshall stability versus Marshall flow index. Figures 18 and 19 are photocopies of two of these stress/strain records. Figure 18 is for a normal mix with a flow index of 13 units of 1/100 inch. Figure 19 illustrates a stress/strain curve for an overasphalted mix with a flow index of 22 units of 1/100 inch. The mix for Figure 19 had rutted badly in service.

It would seem doubtful that the point at which the load "begins to decrease", can be accurately detected when using the usual sleeve type of flow meter. To measure the flow index with sufficient accuracy would appear to require that the Marshall equipment must be equipped with a stress/strain recorder.

If the flow index were measured acurately at hot-mix plants as part of the control procedure during pavement construction, there would be far fewer examples of pavements that are distorted, or flushing and bleeding or both.

V. PAVING ASPHALT SELECTION

1. The Need for Temperature Susceptibility Requirements in Paving Asphalt Specifications

The writer has written extensively on this topic in the past (7,8,9, 10 and 11), but because of space limitations must restrict the subject matter on this topic in this paper.

With respect to pavement performance and pavement design, the two most important properties of a bitumen are:

1. its penetration at 25°C
2. its temperature susceptibility.

Pavement performance and pavement design depend on both of these properties. To try to explain pavement performance, for example, on the basis of penetration at 25°C alone, without temperature susceptibility, is like sending a boxer into the ring with one hand tied behind his back against a very powerful opponent. He will end up on the canvas. However, untie his hand and he will floor his opponent. Similarly, with both penetration and temperature susceptibility one can explain pavement performance and can also design pavements to eliminate or greatly reduce low temperature transverse pavement cracking, to provide adequate warm weather stability, to prevent rutting, and to design recycled pavements on a sound engineering basis.

Currently, neither the Canadian Government Standards Board, AASHTO, nor ASTM have temperature susceptibility requirements in their specifications for paving asphalt. Most Canadian provinces are well ahead of the pack in this respect, and do have some measure of temperature susceptibility in their paving asphalt specifications.

Figures 20 and 21 illustrate what is meant by one measure of temperature susceptibility, pen-vis number (PVN), and how it is measured.

All three bitumens in Figure 20 have the same consistency (penetration or viscosity) at 25°C, but have quite different consistencies at temperatures above and below 25°C. Asphalt 1 is the least temperature susceptible, because its consistency changes the least for a given change in temperature, and it is said to have low temperature susceptibility. The consistency of Asphalt 3 changes the most with a given change in temperature. It will be harder at any given temperature below 25°C and more fluid at any given temperature above 25°C than Asphalts 1 or 2, and it is said have high temperature susceptibility. Asphalt 2 is intermediate between Asphalts 1 and 3 and it is said to have medium temperature susceptibility.

When temperature susceptibility is expressed in terms of pen-vis number (PVN), a bitumen's PVN value can be easily determined from Figure 21 as soon as its penetration at 25°C and its viscosity in centistokes at 135°C have been measured. By plotting these values as the coordinates of a point on Figure 21 (for which the abscissa is penetration at 25°C and the ordinate is viscosity in centistokes at 135°C), its PVN can be easily determined by interpolating between the oblique lines representing different PVN values. On Figure 21, paving asphalts with a high PVN value (PVN = 0.0) have a low temperature susceptibility, while asphalts with a low PVN value (PVN = -1.5) have a high temperature susceptibility.

Tables 4 and 5 and Figures 22 and 23 (11,12,13 and 14) all indicate that the PVN of a paving asphalt, while a measure of its temperature susceptibility, also appears to be a finger print that remains essentially unchanged throughout a pavement's service life.

Anderson and associates from Penn State University (11) show in Figure 22 that the PVN values of the original bitumens and of the thin-film oven test residues from these bitumens, are the same. Table 4 demonstrates that Kandhal and Koehler of PenDot (12) found that even after seven years in service, the PVN of each bitumen recovered from six different pavement sections was practically the same as that of the original asphalt. In 1986, the writer (13) had samples taken of the bitumen being fed into the hot-mix plant and of paving mix being discharged, at nine hot-mix plants around Ontario. PVN values were determined for the original bitumen, for its thin-film oven test residue and for the bitumen recovered from the pavement samples. Table 5 shows that all of these PVN values were very nearly the same. Professor Haas from the University of Waterloo and associates (14) tested samples of pavements from 26 airports across Canada, some of them more than 30 years old, Figure 23. The PVN values for the bitumens recovered from these pavement samples were found to be very near to what the PVN values for the original bitumens must have been.

The reason for these constant PVN values regardless of years of service, is that the hardening of a bitumen in pavement service is an entirely different phenomenon from the oxidation of bitumen that occurs when air is blown through a soft bitumen at 500°F for several hours to produce roofing asphalts with their higher softening points or higher viscosities at 135°C for any given penetration at 25°C. The PVN of a bitumen is substantially constant throughout a pavement's service life, because the gradual hardening of the bitumen does not result in a Group C bitumen oxidizing to a Group B bitumen, or a Group B bitumen oxidizing to a Group A, Figure 23, as would follow if the hardening of a bitumen in pavement service were due to the oxidation phenomenon that produces roofing asphalts.

Anyone who doubts this can quickly investigate it for himself by going to several hot-mix plants in his vicinity, and obtaining samples of the bitumen going into the hot-mix plant, and of the paving mixture being discharged. PVN values can be obtained by running penetration at 25°C and viscosity at 135°C on the original bitumen, on its thin-film oven test residue and on the bitumen recovered from the paving mixture sample. My experience indicates that the three PVN values will be very nearly the same.

The conclusion that a bituminous binder's PVN appears to be a finger print that remains unchanged throughout a pavement's service life forms the basis for Figure 24, which provides a temperature susceptibility chart that in my opinion should be a part of every paving asphalt specification.

Figure 24 is a clear visual chart on which penetration at 25°C and viscosity at 135°C for any paving asphalt in the world can be plotted as the co-ordinates of a point. One can see at a glance how any particular paving asphalt compares with all other paving asphalts. It will fall into one of three groups of temperature susceptibility, Group A, B or C. Furthermore, Figure 24 also shows immediately whether any paving asphalt in question should be used for heavy traffic (low temperature susceptibility, Group A), medium traffic (medium temperature susceptibility, Group B), or light traffic (high temperature susceptibility, Group C).

The Western Canadian Provinces use what they call "windows", consisting of a range of penetration at 25°C and a corresponding range of viscosity, to illustrate their paving asphalt specifications. It should be evident that these "windows" can be easily drawn on Figure 24.

Claims are sometimes made that a temperature susceptibility requirement in a paving asphalt specification would exclude paving asphalts of known good pavement performance. It should be crystal clear that no paving asphalt would be excluded by Figure 24.

The arrows in Figure 24 indicate, as a result of the finger print effect, that even after many years of service, an original Group A bitumen remains essentially as Group A, an original Group B bitumen remains essentially Group B and an original Group C bitumen remains essentially as Group C. The double headed arrows in Figure 24 also indicate for the bitumen recovered from an old pavement, the direction in which it has hardened in service, and what its original penetration at 25°C may have been. This could be very useful for the design of recycled paving mixtures, which will be more and more important from now on.

2. Criteria for Paving Asphalt Selection

In cold climates, when designing dense graded bituminous paving mixtures, three basic objectives should be kept clearly in mind:

1. Avoiding low temperature transverse pavement cracking in winter.

2. Providing for adequate stability for fast traffic in summer.

3. Preventing pavement rutting under traffic in warm weather.

Every pavement should satisfy all three of these design requirements.

In climates without frost, only items 2 and 3 above are of concern.

Figures 25 and 26 indicate how paving asphalts should be selected to avoid low temperature transverse pavement cracking in winter and to provide adequate pavement stability for summer traffic.

Figure 25 is based on Test Road data (7,8,9,10 and 11), on theoretical considerations (15), and on observations of the field performance of many hundreds of miles of paved highways, and represents the writer's best estimate of the combinations of penetration at 25°C and of viscosity at 135°F for the original bitumens to be selected. These will eliminate or at least greatly reduce low temperature transverse pavement cracking at the minimum winter temperature at a pavement site, throughout a pavement's service life. For example, if the minimum pavement temperature anticipated at a pavement site is -28.9°C (-20°F), only combinations of penetrations at 25°C and viscosities at 135°C that lie on or to the right of the oblique line in Figure 25 that is labelled -28.9°C (-20°F) should be selected if the pavement is to avoid low temperature transverse cracking at -28.9°C (-20°F) throughout its service life. Grades of paving asphalt that lie to the left of this line are too hard, and would result in thermal cracking at the anticipated minimum temperature of -28.9°C (-20°F). This is also true of each of the other oblique lines in Figure 25 representing other anticipated minimum winter pavement temperatures.

On the other hand, it should be noted that to obtain the highest pavement stability in warm weather, the combinations of penetration at 25°C and of viscosity at 135°C that lie on each pertinent oblique temperature-labelled line should also be selected, because these will provide the lowest penetrations at 25°C and therefore the highest pavement stabilities that can be attained without causing low temperature transverse pavement cracking.

Figure 26 is based in part on Figure 25 and emphasizes that for a paving site where the expected minimum winter pavement temperature will be -23.5°C (-10°F), for example, paving asphalts of low temperature susceptibility should be selected for thin pavements or for surface courses that will be subjected to heavy traffic, because these bitumens have the lowest penetrations at 25°C, and will therefore provide the highest pavement stability, and at the same time will just avoid low temperature transverse pavement cracking (7,8,9,10 and 11). For medium and light traffic, bitumens of medium and high temperature susceptibility can be used because as Figure 26 demonstrates, when these bitumens are right on the oblique line labelled -23.3°C (-10°F), pavements containing them will avoid low temperature transverse pavement cracking, and although because of their higher and higher penetrations at 25°C they provide lower and lower pavement stabilities (moduli of stiffness), these stabilities are still adequate for medium and light traffic, respectively (7,8,9,10 and 11).

Reference has been made elsewhere to the increase in pavement temperature with pavement depth below the surface on the coldest dayof the year, and the decrease of pavement temperature with pavement depth on the hottest day of the year (7,8). It has also been shown elsewhere (7), that in regions where bitumens of low temperature susceptibility are scarce, bitumens of medium and high temperature susceptibility can be selected for binder or base courses, with the limited bitumens of low temperature susceptibility being reserved for the surface course. With this composite structure, the overall pavement can be designed to continue to avoid low temperature transverse pavement cracking.

The Marshall method will continue to be used for many years as a measure of pavement stability, and will only be slowly replaced by measurements of the modulus of stiffness, which is the measure of pavement stability utilized in many of the writer's papers (9,10). Consequently, Figures 25 and 26 are highly useful as a guide to the selection of the bitumens to be used for paving mixture design by the Marshall method.

Table 2 provides criteria for different traffic categories that the writer favours for the Marshall method of design.

Table 3 contains criteria for the Marshall test method of design for different traffic categories that are recommended by The Asphalt Institute.

VI. DESIGNING PAVING MIXTURES WITH GREATER RESISTANCE TO RUTTING

Asphalt pavement rutting, Figure 27, has become a world-wide problem. While pavement rutting is always highly visible as a surface phenomenon, Figure 28 shows that pavement foundation subsidence due to further compaction of the subgrade or granular base or both by traffic, or lateral displacement of the subgrade or base course because one or both are overloaded by the traffic being carried, can be an unseen major contributing factor. However, for pavements on strong foundations, rutting can be due to additional compaction of the pavement itself by traffic, or to lateral displacement because of underdesign or overloading of the pavement.

Paving asphalts exhibit two quite different properties, elastic and viscous, depending upon pavement temperature and time of flooding by traffic. Under very fast loading, a point on a pavement surface is exposed to a time of loading of about 0.01 second by a heavy truck tire travelling at 100 km per hour. Under this very rapid loading, even at summer temperatures, the asphalt binder approaches purely elastic behaviour (obeying Hooke's Law). That is, the asphalt pavement deflects momentarily under the load applied, but rebounds to, or approximately to, its original elevation when the wheel has passed.

Under much slower or stationary traffic, such as in a parking area, the paving asphalt's viscous property becomes important and a pavement can begin to deform or rut under load.

The viscosity of a normal paving asphalt depends chiefly upon its crude oil source, but partly upon its method of manufacture, and its possible increase in viscosity is therefore severely limited. However, as pointed out by the small circles in Figure 29, by the addition of a small percentage of a proper polymer, the viscosity of a paving asphalt can be increased several times, thereby providing sufficient viscous resistance to eliminate or at least greatly reduce pavement rutting (11).

VII. SELECTING THE OPTIMUM BITUMEN CONTENT ON CLIMATES WITHOUT FROST

Figure 30 illustrates the proper selection of bitumens for pavements in climates for which there is no frost (11).

At present, paving asphalt for any paving project in these climates is normally specified as a single grade, eg. 80/100 penetration, without the slightest regard for the bitumen's temperature susceptibility.

Figure 30 demonstrates that in this case, the pavement stability can range from a modulus of stiffness of 7,000 psi when the pavement contains 100 penetration bitumen with a high temperature susceptibility, PVN = -1.5, to a modulus of stiffness of 18,000 psi, if the pavement contains 80 penetration bitumen with a low temperature susceptibility, PVN = 0.0. These pavement stabilities are developed by fast heavy truck traffic travelling at 100 km/hr (60 miles/hr), when the pavement temperature is 60°C (140°F). It is obvious that if the pavement required a bitumen with a penetration at 25°C of 80, and a low temperature susceptibility, PVN = 0.0, to develop a needed modulus of stiffness of 18,000 psi at 60°C (140°F), the pavement stability and serviceability will not be very satisfactory if it contains a bitumen with a penetration at 25°C of 100 and a high temperature susceptibility, PVN = -1.5, that develops a pavement modulus of stiffness of only 7,000 psi for the same pavement temperature and rate of loading. Nevertheless, in the absence of temperature susceptibility requirements in paving asphalt specifications, this is what is happening.

As illustrated by Figure 30, one solution for this problem requires that as the temperature susceptibility of the bitumen increases, its penetration at 25°C must be decreased in order to provide a constant pavement stability (modulus of stiffness) as follows:

(a) 80/100 penetration bitumen, if its PVN is 0.0
(b) 70/86 penetration bitumen, if its PVN is -0.5
(c) 60/76 penetration bitumen, if its PVN is -1.0
(d) 50/65 penetration bitumen, if its PVN is -1.5

VIII. ELIMINATING OR GREATLY REDUCING PAVEMENT RUTTING IN CLIMATES WITHOUT FROST

While the solution just described should take care of the problem of providing constant stability it may not eliminate or greatly reduce any pavement rutting that may be occurring, and it may even increase the tendancy for rutting. It was pointed out earlier that bitumen has two different properties. Under very fast loading even at summer temperatures, it exhibits or approaches purely elastic behaviour. Under a very slow rate of loading, as in a parking area, it can exhibit purely viscous behaviour, and can creep or rut (11).

Consequently, to eliminate or greatly reduce rutting, the viscosity of the bitumen has to be increased. As shown by the small circles in Figures 29 and 30, the addition of a small percentage of a proper polymer can increase the viscosity several times and thereby greatly reduce or eliminate rutting. In Figures 29 and 30, the lowest small circles represent the viscosity of a normal asphalt, while the higher circles illustrate the increase in viscosity that can be provided by the incorporation of small percentages of a proper polymer (11).

At the same time, as indicated by Figures 29 and 30, the pavement stability (modulus of stiffness) under fast traffic in warm weather, can also be substantially increased by the addition of a small percentage of a suitable polymer.

IX. PAVEMENT RECYCLING

Avoidance of low temperature transverse pavement cracking is as important after pavement recycling, as it should have been for the original pavement. Suitable guidelines to achieve this do not appear to be in use at the present time. However, recognition of paving asphalt temperature susceptibility seems to offer a promising approach to this problem, as illustrated in Figure 31, for pavement recycling in cold climates (11).

The background for Figure 31 is Figure 25, which illustrated the selection of bitumens for original paving mixtures to avoid low temperature transverse pavement cracking. For Figure 31, it is assumed that the minimum winter pavement temperature at Location 1 in some given area is -28.9°C (-20°F). Location 1 is the site of the proposed recycling project because the existing pavement shows serious low temperature transverse pavement cracking and other evidence of severe pavement distress.

Ordinarily, everyone will have forgotten what the characteristics of the original asphalt binder were when the badly cracked pavement at Location 1 was constructed many years ago, which upon extraction is found to have a penetration of 20 at 25°C and a viscosity at 275°F of 820 centistokes, (PVN = -0.9). This indicates that the asphalt binder in this pavement is of medium temperature susceptibility, or in the Group B, category.

It is assumed that the recycled pavement is to carry much more traffic, and that it is to be designed for the heavy traffic category in Figures 24, 25 and 31. For the middle of the heavy traffic band (PVN = -0.25), and for a minimum winter pavement temperature of -28.9°C (-20°F), Figure 31 indicates that this would require a bitumen of 180 penetration at 25°C and a viscosity at 135°C of 250 centistokes (PVN = 0.25), and would be in the Group A traffic category (Point 2). This would be provided by Treatment A, (Line 2) consisting of a single or a combination of softening agents that would change the penetration from 20 at 25°C, and a viscosity of 820 centistokes at 135°C, in the old pavement, to a penetration of 180 at 25°C and a viscosity of 250 centistokes at 135°C in the recycled paving mixture. A large assortment of softening agents, including soft asphalts and commercial modifiers is available to the designer for this purpose.

After this treatment, according to the finger print effect, the asphalt binder in the recycled pavement would slowly harden in service as indicated by Line 3 in the heavy traffic category of Figure 31.

The engineer responsible for the recycled pavement project might decide that Treatment A resulting in an asphalt binder of 180 penetration at 25°C and a viscosity of 250 centistokes at 135°C (PVN = -0.25), would not have the required minimum stability at summer temperatures for the anticipated heavier traffic. He might therefore, favour Treatment B, for example, for the recycled pavement to provide a bitumen with a penetration at 25°C of 120 and a viscosity at 135°C of 575 centistokes (PVN = +0.5), Point 4. This would also avoid low temperature transverse pavement cracking at -28.9°C (-20°F) throughout the pavement's service life, if it were properly designed and constructed.

It would be difficult to locate a normal combination of softening agents that could change the penetration of 20 at 25°C for the bitumen in the old pavement to 120 penetration at 25°C, and at the same time increase the bitumen's PVN from -0.9 to +0.5, because a PVN of +0.5 is outside of the range of PVN for nearly all bitumens used for paving. However, a PVN of +0.5 could be easily obtained by the addition of a small percentage of an appropriate polymer (11).

Following Treatment B, and in accordance with the finger print effect, the bitumen in the recycled paving mixture could be expected to gradually harden in service along Line 5 within the heavy traffic portion of Figure 31.

A similar method would be required for pavement recycling for other minimum winter pavement temperatures illustrated in Figure 31.

Figure 32 indicates a similar approach to the design of recycled paving mixtures for a given region that is free from frost (11). For the bitumen recovered from an old pavement, by determining its penetration at 25°C and its viscosity at 135°C, and plotting these data as a point on Figure 32, the temperature susceptibility (Group A, B or C) to which the bitumen in the old pavement belongs, can be very quickly established. The background of Figure 32 is that of Figure 24.

Suppose for example, the recovered bitumen has a penetration of 20 at 25°C, and that it lies in Group B (PVN = -0.5 to -1.0), which is suitable for medium traffic, Point 1. Because of increased traffic on this old pavement, the engineer has decided to design the recycled pavement for the heavy traffic category for which a Group A bitumen is required, for example 120 penetration at 25°C. By adding an appropriate softening agent or group of softening agents (Treatment A) the bitumen of 20 penetration and Group B can be softened to a Group A asphalt of 120 penetration as shown by Point 2 on Figure 32. After reconstruction the bitumen in the recycled pavement will remain in Group A and will harden in service along Line 3, if it conforms to the finger print principle.

However, after further consideration, the engineer may decide that he requires a harder bitumen of 85 penetration with a still lower temperature susceptibility (higher PVN), for the recycled paving mixture to carry the anticipated traffic loading. In this case, starting with the recovered bitumen of 20 penetration from the old pavement, and incorporating the softening material or materials indicated by Treatment B, he may soften the 20 penetration bitumen recovered from the old pavement to a penetration of 85 at 25°C and with a PVN of +0.3, Point 4 on Figure 31. However, because a PVN of +0.3 is higher than normal paving bitumens or other softening agents can provide, he may have to incorporate a small percentage of a suitable polymer to achieve a PVN of +0.3. This may also be necessary for Treatment A. After Treatment B, the bitumen in the recycled paving mixture can be expected to harden in service along Line 5 (finger print effect) (11).

Because of the addition of the required softening materials, the bitumen content of a 100 percent recycled paving mixture would probably be too high for adequate stability and resistance to rutting, and the incorporation of new aggregate would most likely be necessary. In this case, the proportion of old pavement in the recycled paving mixture would have to be reduced. This however, would have the advantage of correcting any deficiency in the gradation of the old pavement to be recycled.

For cold mix pavement recycling, Treatments A or B, etc., can be applied in the form of an emulsion.

Figure 33 indicates that about six months time is required for an asphalt emulsion to thoroughly cure and to develop the pavement stability that its base asphalt would develop if it were used in a hot paving mixture (17). Figure 34 provides data on cured Types I, II and III emulsified asphalt paving mixtures made with increasingly inferior aggregates. The stabilities of the fully cured asphalt emulsion mixes are shown to compare favourably with corresponding hot mixes.

SUMMARY

1. The Asphalt Institute method of design is recommended for dense graded asphalt concrete paving mixtures using 3 to 5 percent air voids and percent minimum VMA values that vary with the nominal maximum aggregate particle size.

2. Air voids are defined as the small pockets of air between the coated aggregate particles in a compacted paving mixture.

3. VMA (voids between the mineral aggregate particles in a compacted paving mixture) is defined as the void space between the aggregate particles when the volume of the aggregate is calculated by means of its ASTM bulk specific gravity.

4. It is shown that minimum VMA values are needed in a compacted paving mixture to ensure adequate space between the aggregate particles for 3 to 5 percent air voids plus the volume of bitumen required for a durable pavement.

5. It is shown that a combination of percent air voids and percent voids filled with bitumen is a good criterion for pavement evaluation, but is much too restrictive for paving mixture design which should be based on percent air voids and percent VMA.

6. A useful definition is given for "nominal maximum aggregate particle size" to use in conjunction with minimum VMA values.

7. The Ontario grading bands for surface course and base course paving mixtures are endorsed.

8. It is shown that when Asphalt Institute design is enforced, there is no advantage in using a high percentage of mineral filler, and there can be disadvantages.

9. PVN as a measure of paving asphalt temperature susceptibility appears to provide a finger print that stays with the paving asphalt throughout the service life of the pavement in which it is incorporated.

10. As a measure of paving asphalt temperature susceptibility, PVN can be used to divide paving asphalts into three different groups of temperature

susceptibility, low temperature susceptibility, Group A, medium temperature susceptibility, Group B and high temperature susceptibility, Group C.

11. Based on their temperature susceptibility values a chart is provided to guide the selection of paving asphalts that will prevent low temperature transverse pavement cracking at each of a wide range of low winter temperatures.

12. Another chart is provided that indicates that bitumens of low temperature susceptibility, Group A, should be selected for heavy traffic, bitumens of medium temperature susceptibility, Group B, should be selected for medium traffic, and bitumens of high temperature susceptibility Group C should be selected for light traffic, all of which in turn would avoid or greatly reduce low temperature transverse pavement cracking.

13. Because bitumens exhibit elastic behaviour under fast loading and viscous behaviour under slow loading, it is suggested that insofar as the bituminous binder is concerned, pavements containing bitumens of higher viscosity will demonstrate less tendancy for pavement rutting.

14. A method is described for the design of recycled paving mixtures in climates with and without frost.

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14. McLeod, Norman W., Discussion of paper by Haas and Associates "A Comprehensive Study of Cold Climate Airport Pavement Cracking", Prepared for Presentation at The Association of Asphalt Pavement Technologists' Annual Meeting, Reno, Nevada, February 23-25, 1987.

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16. McLeod, Norman W., "The Case for Grading Asphalt Cements by Penetration at 25°C (77°F)", Proceedings, Canadian Technological Asphalt Association, Vol. 20, 1975, p. 251.

17. The Asphalt Institute, "Research and Development of The Asphalt Institute's Thickness Design Manual (MS-1) Ninth Edition", Research Report No 82-2, August 1982.

Table 1

Voids Properties of Compacted Paving Mixtures

Specific Gravity of a compacted paving mixture = 2.401
Theoretical max. spec. gravity of the paving mixture = 2.485
Asphalt content of paving mix, (lb asphalt per 100 lb mix) = 6.0
Asphalt specific gravity = 1.025

Agg. Specific Gravity	Air Voids %	VMA %	% Voids Filled	% Asphalt Absorp. Based on Tot. Agg.	% Water Absorp. Based on Tot. Agg.
ASTM bulk - 2.675	3.4	15.6	78.2	0.82	1.59
Virtual (effective) - 2.734	3.4	17.5	80.3	Nil	1.59
ASTM apparent 2.794	5.2	19.2	72.9	Nil	1.59

Table 2(a)

Physical Requirements for Surface Course Paving Mixtures

Property of Laboratory Compacted Mixtures	AADT for 2 Lanes					
	>5000		1000 to 5000		<1000	
Compactive Effort	75-blows Hand Compactor		75-blows Hand Compactor		75-blows Hand Compactor	
	Min	Max	Min	Max	Min	Max
Marshall Stability N at 60°C	8900	-	6700	-	4400	-
Marshall Flow Index Units of 0.25 mm	9.0	18	9.0	16	9.0	16
% Air Voids	3	5	3	5	3	5
% VMA	See Fig.7	-	See Fig.7	-	See Fig.7	-
Optimum Bitumen Content	4.5% air voids		4.0% air voids		3.5% air voids	

Table 2(b)

Physical Requirements for
Base Course Paving Mixtures

Property of Laboratory Compacted Mixtures	AADT for 2 Lanes					
	>5000		1000 to 5000		<1000	
Compactive Effort	75-blows Hand Compactor		75-blows Hand Compactor		75-blows Hand Compactor	
	Min	Max	Min	Max	Min	Max
Marshall Stability N at 60°C	8000	-	5800	-	4400	-
Marshall Flow Index Units of 0.25 mm	9.0	18	9.0	16	9.0	16
% Air Voids	3	5	3	5	3	5
% VMA	See Fig.7	-	See Fig.7	-	See Fig.7	-
Optimum Bitumen Content	4.5% air voids		4.0% air voids		3.5% air voids	

Table 3

Asphalt Institute Marshall Design Criteria

Traffic Category No of Compaction Blows Each End of Specimen	Heavy 75		Medium 50		Light 35	
Test Property	Min	Max	Min	Max	Min	Max
Stability, all mixtures (N)	750	-	500	-	500	-
Flow, all mixtures 0.01 in (0.25 mm)	8	16	8	19	8	20
Percent Air Voids			3	5	3	5
Surface of Leveling	3	5				
Base	3	8	3	8	3	8
Percent Voids in the Mineral Aggregate	See Fig.7		See Fig.7		See Fig.7	

TABLE 4 TEMPERATURE SUSCEPTIBILITIES OF ORIGINAL AND AGED ASPHALTS

ASPHALT TYPE	PI (pen/pen)				PVN (Pen-Vis Number)			
	Original	Just After Construction	20 Months	Seven Years	Original	Just After Construction	20 Months	Seven Years
T-1	-2.77	-2.24	+0.36	+1.82	-1.04	-1.13	-1.07	-1.12
T-2	-0.71	-0.80	+1.22	+1.52	-0.70	-0.68	-0.54	-0.60
T-3	-1.51	-0.99	-0.12	-0.58	-0.61	-0.72	-0.65	-0.56
T-4	-1.05	-0.65	+0.93	+0.39	-0.86	-1.03	-0.76	-0.79
T-5	-2.23	-2.03	-0.32	-0.87	-1.03	-1.16	-1.07	-1.12
T-6	-1.29	-0.64	+0.60	-0.46	-0.45	-0.47	-0.40	-0.39

Table 5

Comparison of PI (Pen/Pen) and PVN Values for Original Paving Asphalts, Their Thin-Film Oven Test Residues, and Recovery After Discharge from a Hot-Mix Plant.

ORIGINAL ASPHALT		PI PEN/PEN (HEUKELOM)			PVN		
NO	PEN	ORIGINAL	THIN FILM RESIDUE	PUGMILL DISCHARGE	ORIGINAL	THIN FILM RESIDUE	PUGMILL DISCHARGE
1	85/100	-2.86	-2.33	-1.81	-0.61	-0.67	-0.56
2	85/100	-1.63	-2.06	-2.00	-0.67	-0.69	-0.69
3	85/100	-2.73	-1.64	-2.18	-0.70	-0.68	-0.67
4	150/200	-1.73	-1.16	-0.65	-0.59	-0.67	-0.64
5	85/100	-1.98	-2.38	-0.81	-0.67	-0.69	-0.56
6	85/100	-1.23	-1.06	-0.84	-0.77	-0.64	-0.49
7	85/100	-0.94	-0.21	-0.80	-0.47	-0.41	-0.36
8	85/100	-1.11	-2.88	-1.93	-0.55	-0.56	-0.47
9	85/100	-1.24	-1.49	-1.92	-0.53	-0.52	-0.34

FIGURE 1: APPEARANCE OF AN ASPHALT PAVEMENT THAT WAS MADE WITH A HARD VOLCANIC PUMICE AGGREGATE

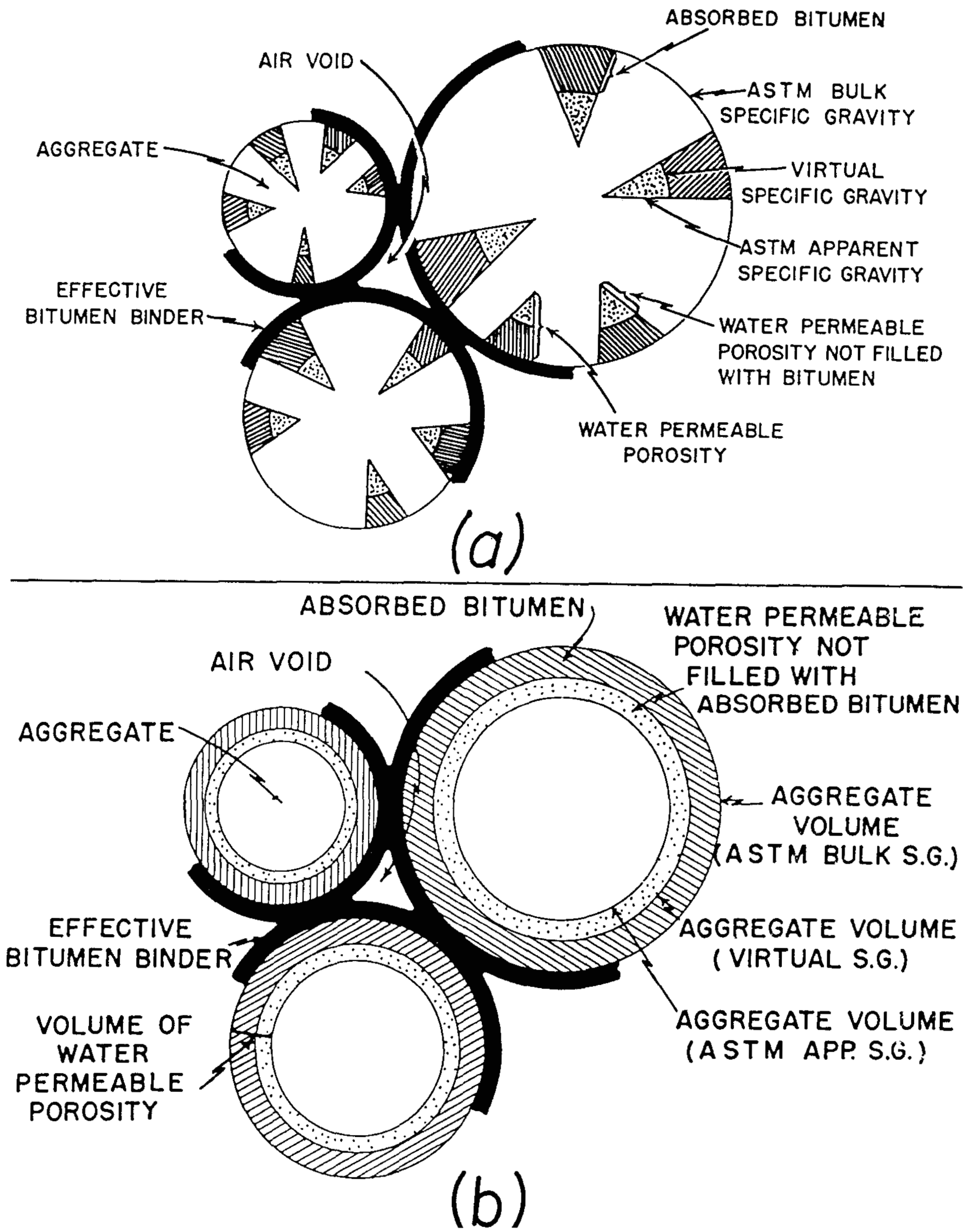

FIGURE 2: ILLUSTRATING INFLUENCE OF ASPHALT ABSORPTION AND THREE DIFFERENT DEFINITIONS FOR AGGREGATE SPECIFIC GRAVITY ON VALUES FOR VMA AND AIR VOIDS FOR COMPACTED BITUMINOUS PAVING MIXTURES.

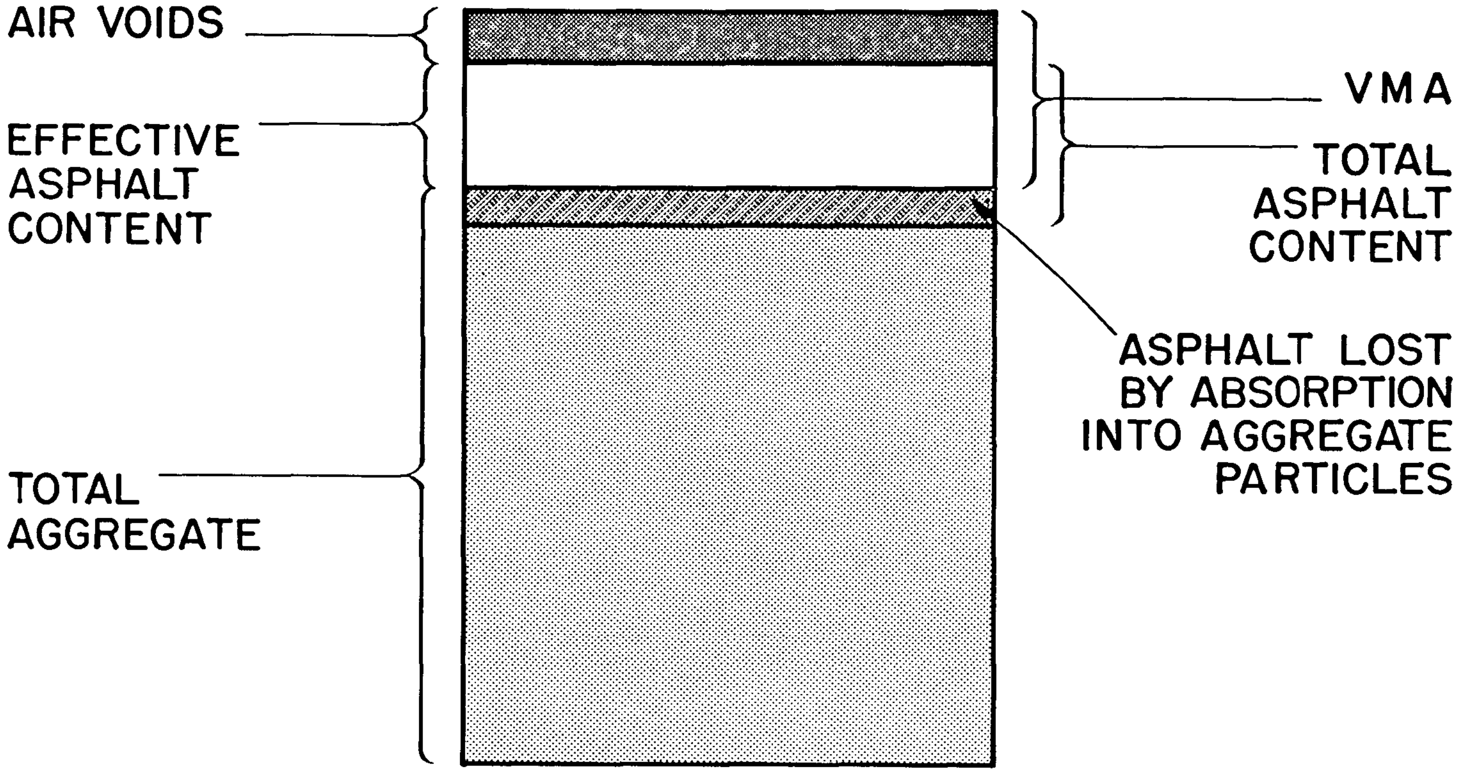

FIG. 3 ILLUSTRATING VOLUME RELATIONSHIPS BETWEEN TOTAL ASPHALT CONTENT, EFFECTIVE ASPHALT CONTENT AND TOTAL AGGREGATE IN A COMPACTED PAVING MIXTURE.

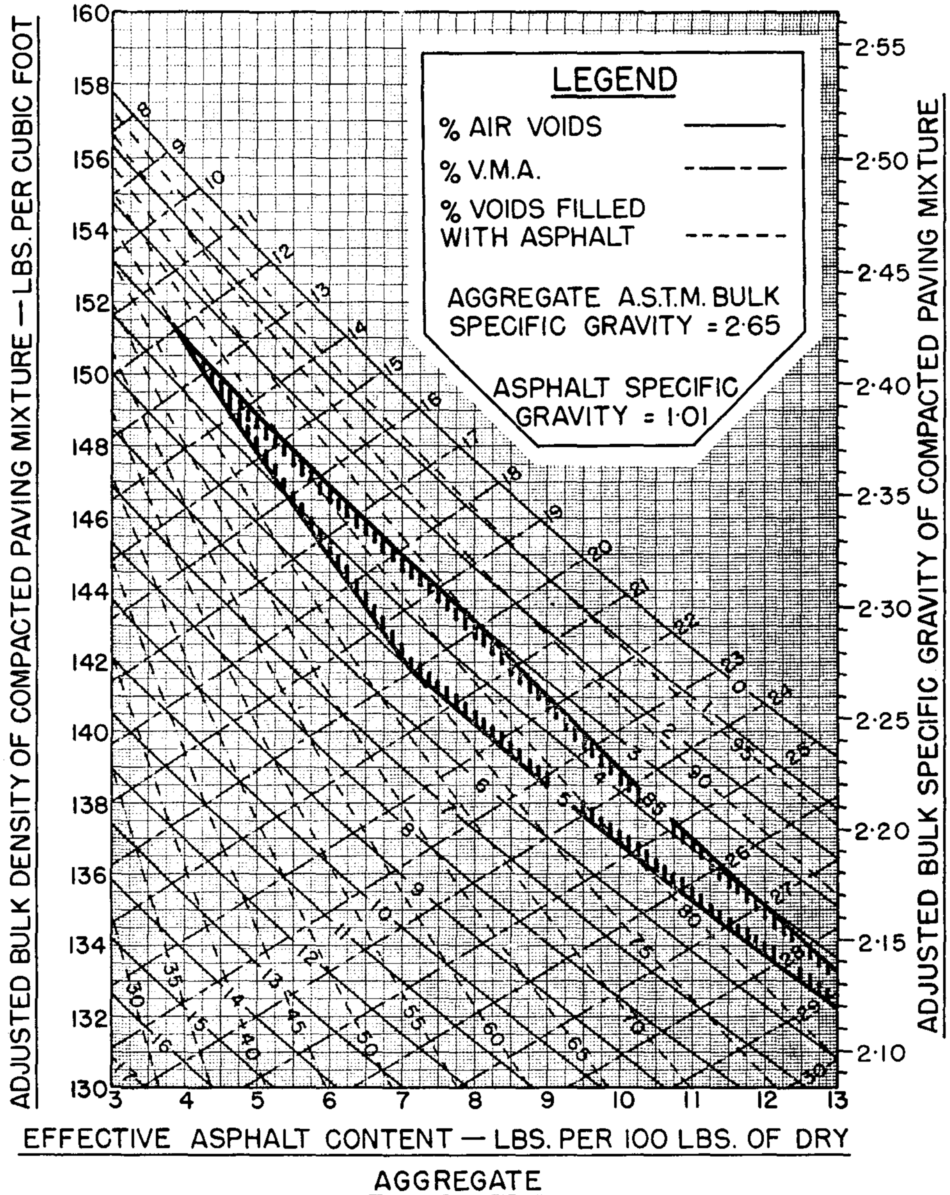

FIG. 4 INFLUENCE OF SPECIFICATION REQUIREMENTS OF 3 TO 5 PER CENT AIR VOIDS AND 75 TO 85 PER CENT AGGREGATE VOIDS FILLED ON THE DESIGN OF PAVING MIXTURES.

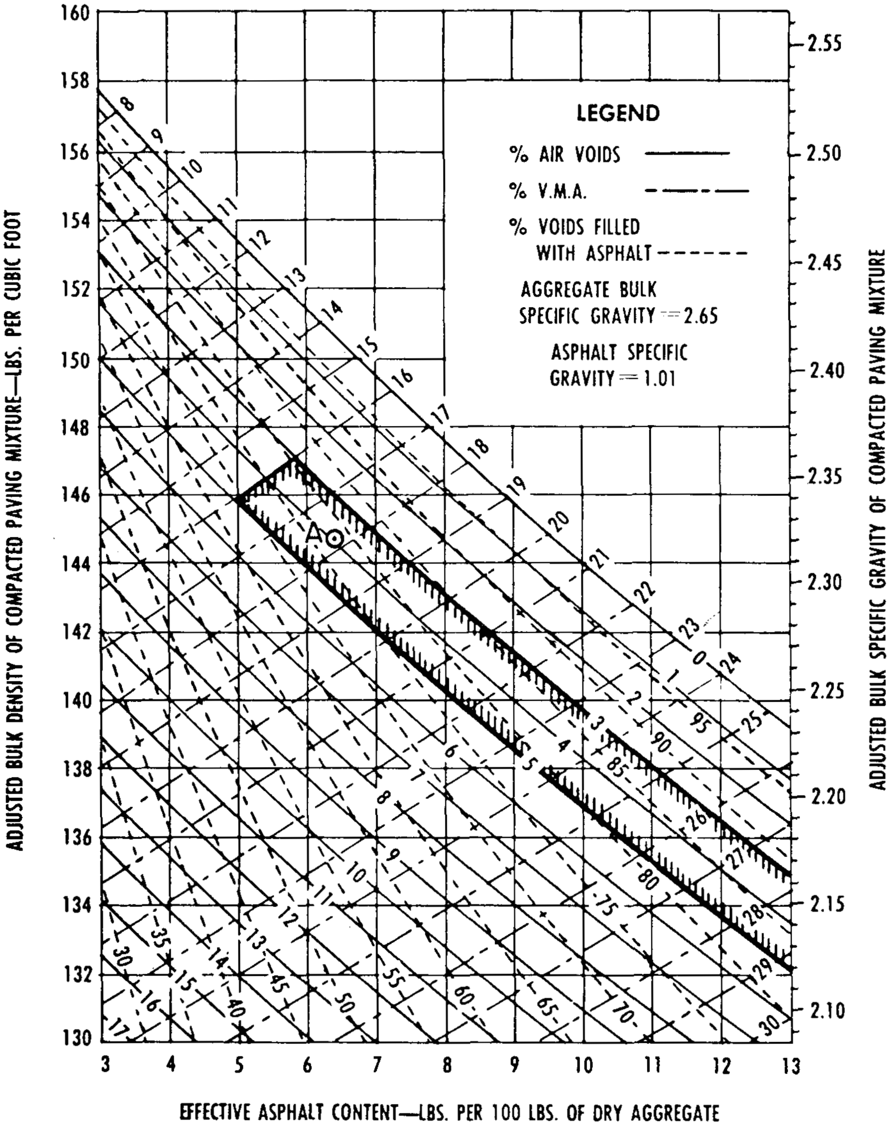

FIG. 5 GRAPHICAL METHOD FOR DETERMINING VMA AND AIR VOIDS IN A COMPACTED PAVING MIXTURE (ASPHALT CONTENT EXPRESSED AS POUNDS PER 100 POUNDS OF DRY AGGREGATE).

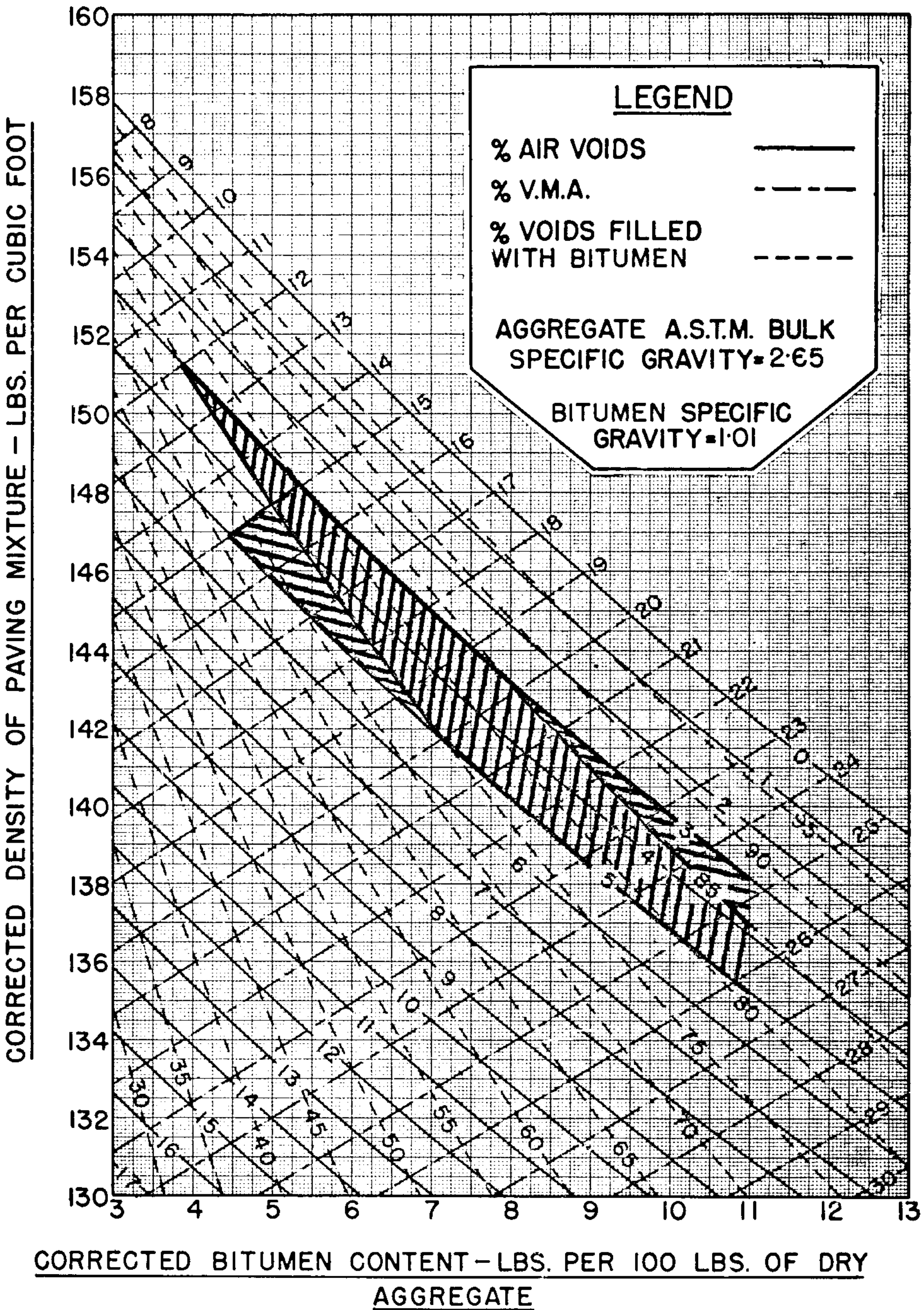

FIG. 6 COMPARISON OF DESIGNS FOR PAVING MIXTURES BASED ON 3 TO 5 PER CENT AIR VOIDS AND 75 TO 85 PER CENT VOIDS FILLED VERSUS 3 TO 5 PER CENT AIR VOIDS AND 15 PER CENT MINIMUM V.M.A.

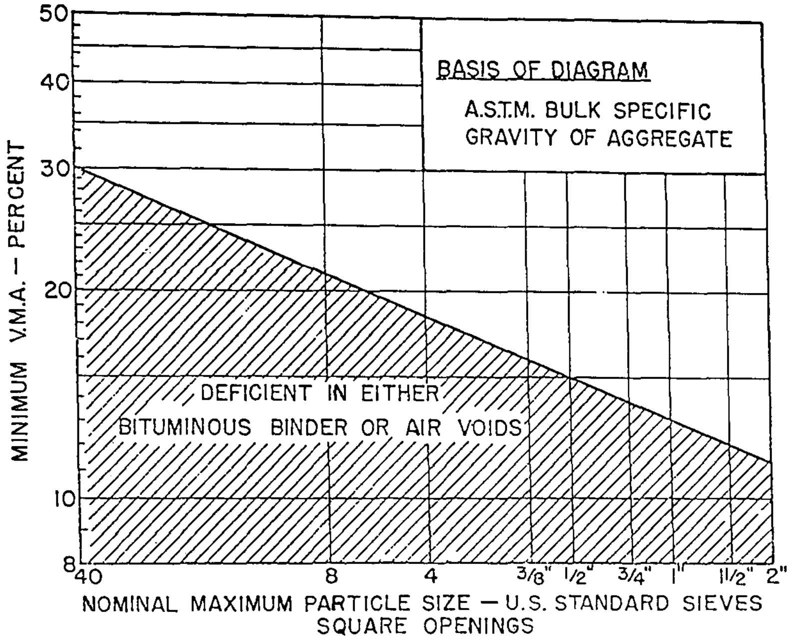

FIGURE 7 RELATIONSHIP BETWEEN MINIMUM V.M.A. AND NOMINAL MAXIMUM PARTICLE SIZE OF THE AGGREGATE FOR COMPACTED DENSE GRADED PAVING MIXTURES.

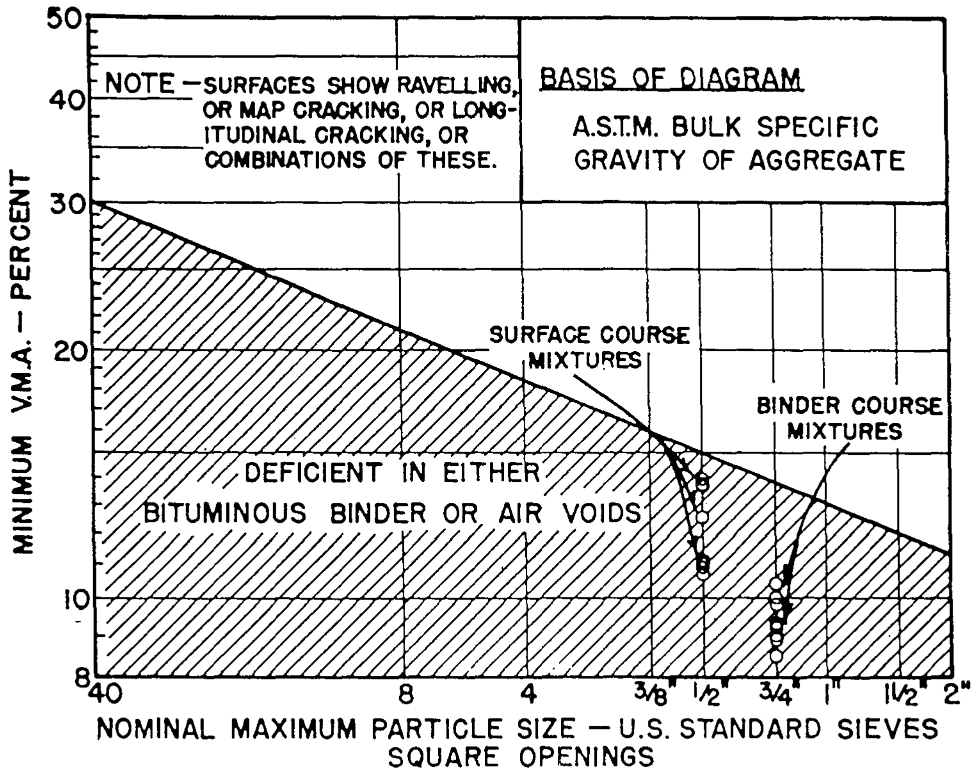

Figure 8 Poor pavement performance because of low VMA values

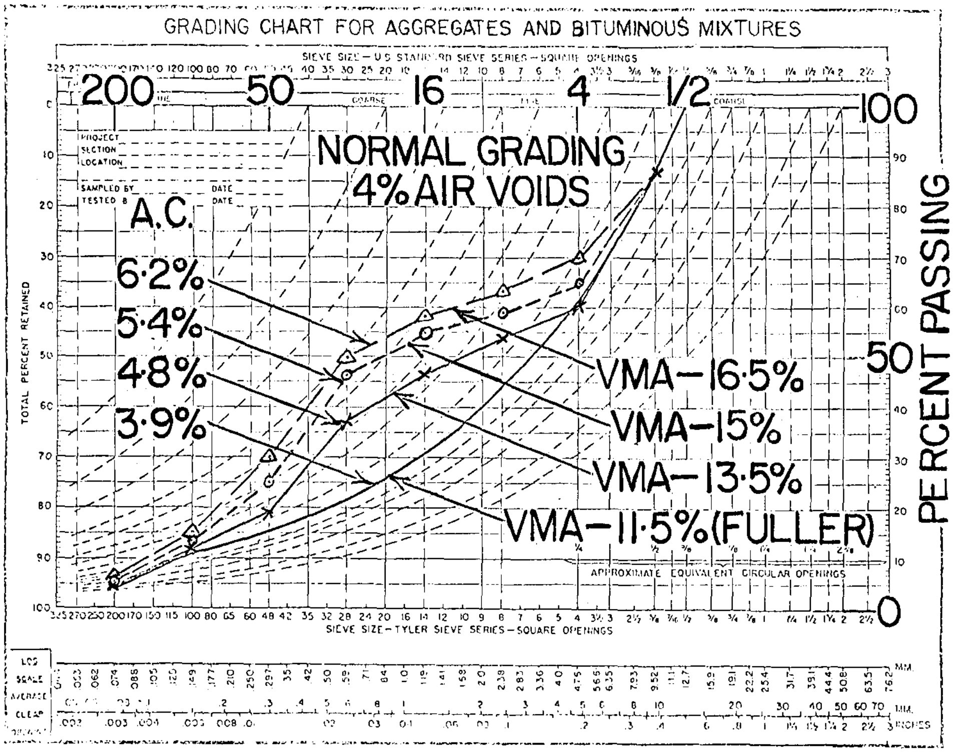

Figure 9 For constant air voids, higher VMA values are obtained by deviating further from the corresponding Fuller curve.

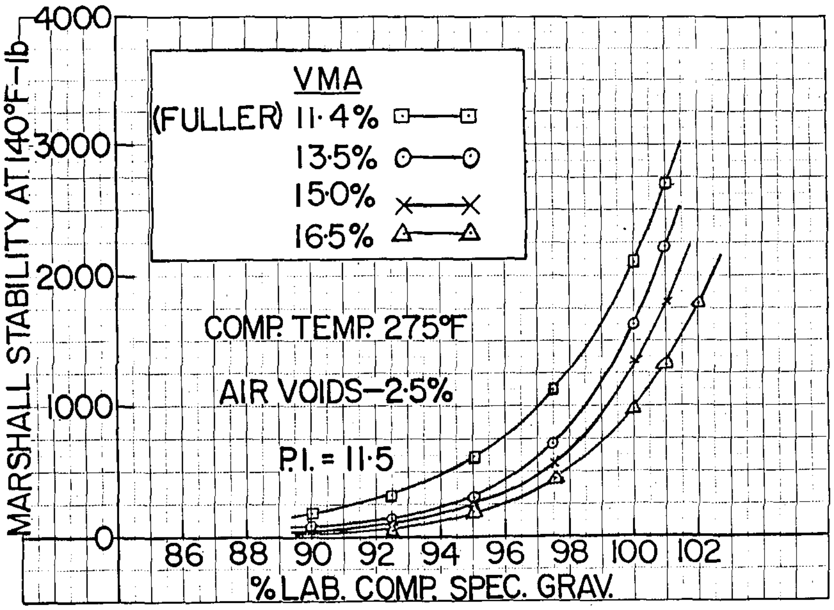

Figure 10 At a constant air voids value Marshall stability increases as VMA decreases.

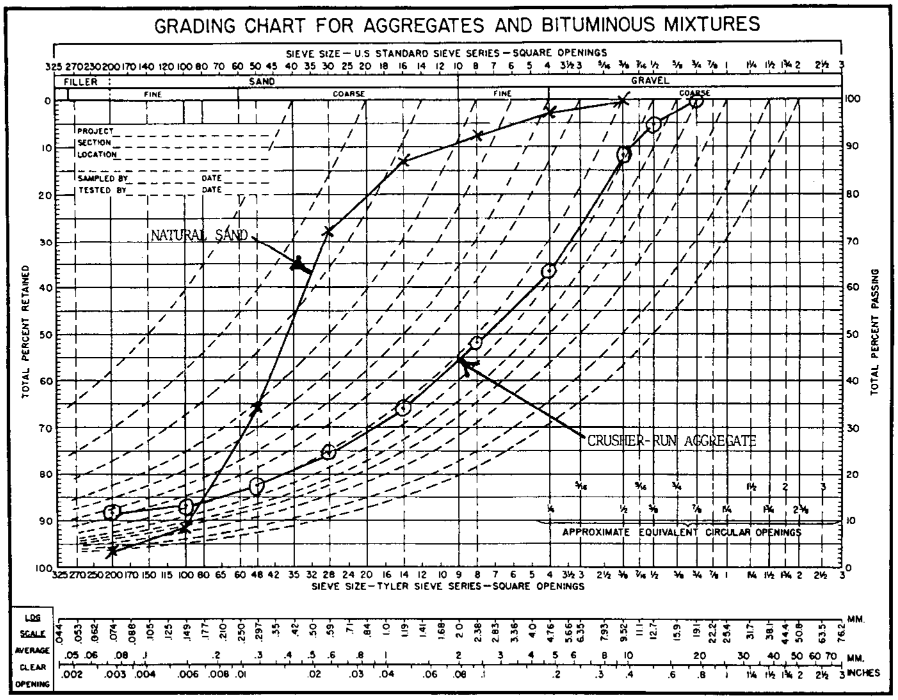

FIGURE 11 ILLUSTRATING GRADING CURVES FOR NATURAL SAND AND CRUSHER-RUN AGGREGATE

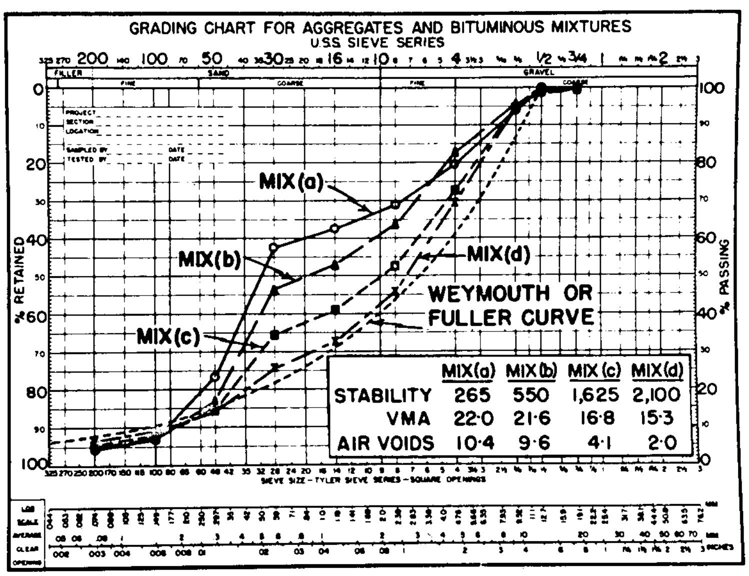

FIG. 12 GRADING CURVES FOR DIFFERENT BLENDS OF COARSE AND FINE AGGREGATES.

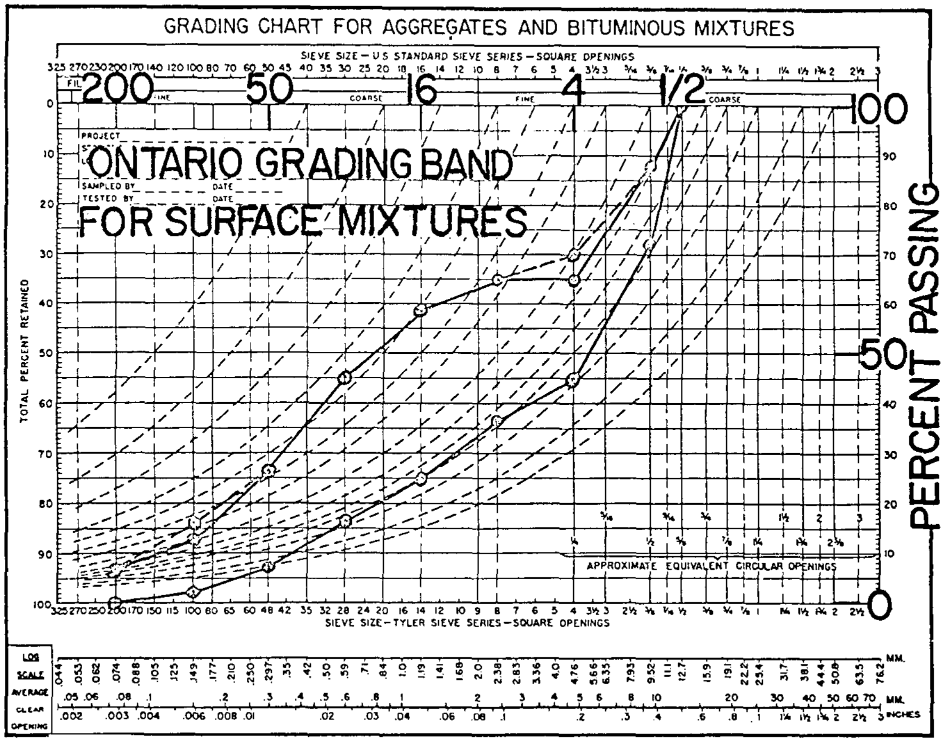

Figure 13 Bulge in Ontario's grading bands makes it possible to satisfy normal VMA requirements.

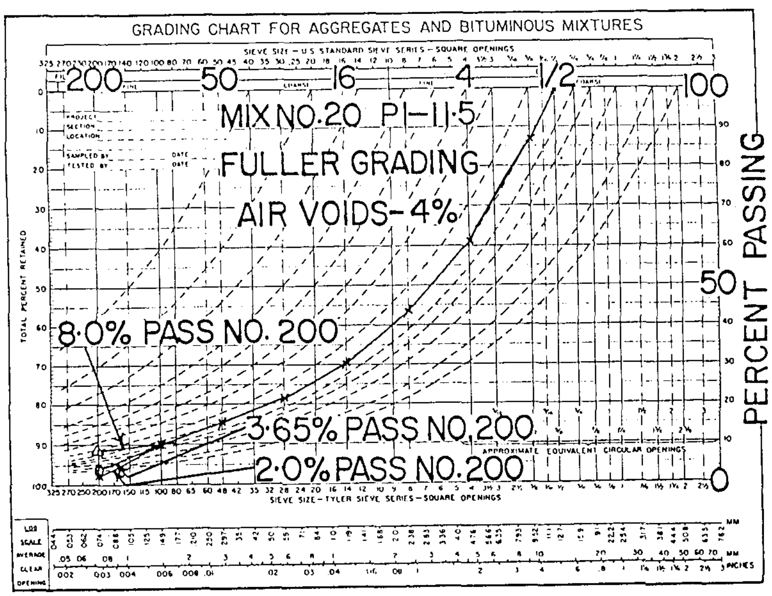

Figure 14 Fuller grading curves for various percentages passing No. 200.

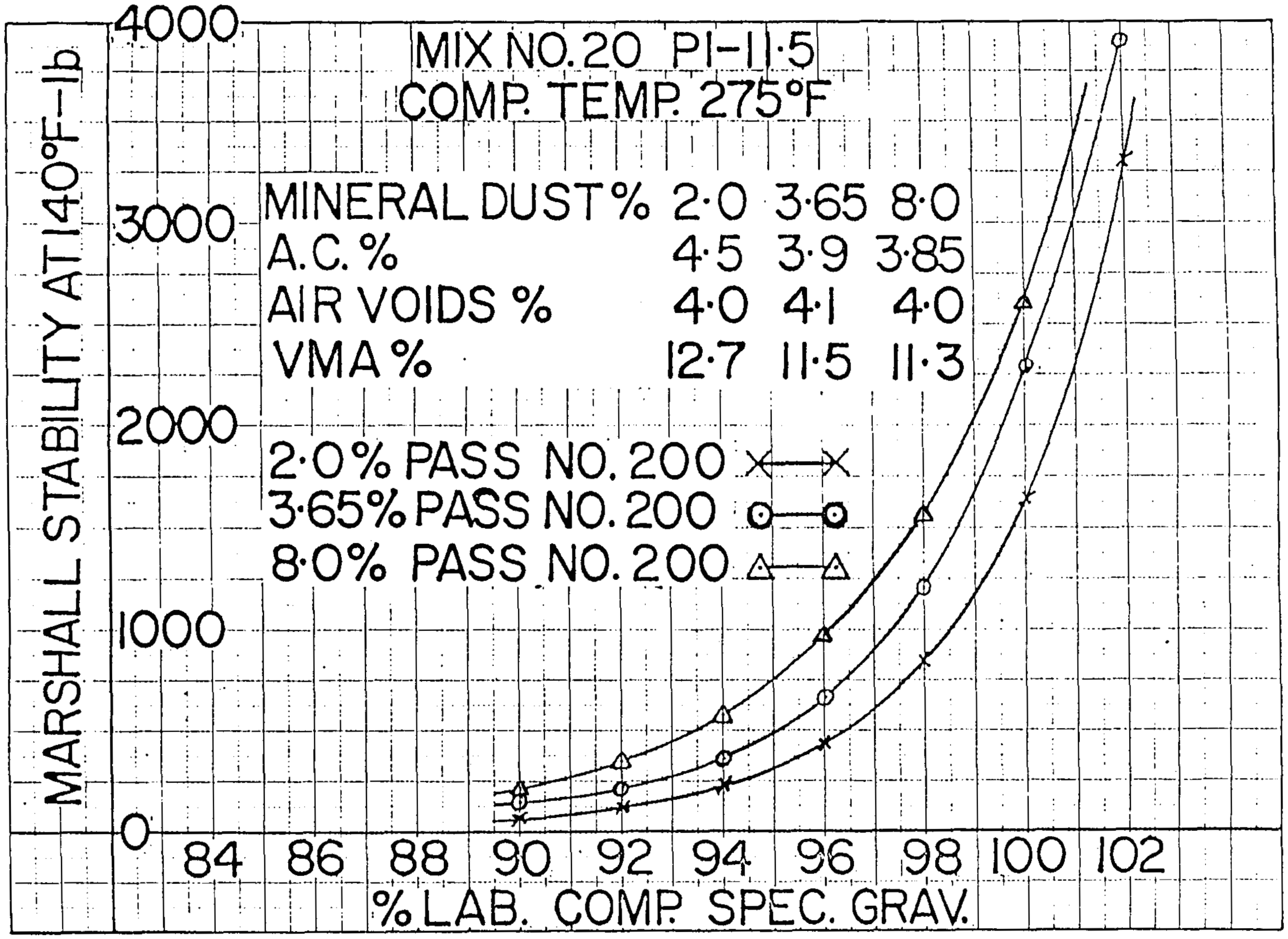

Figure 15 Illustrating increase in Marshall stability with an increase in percent passing No. 200.

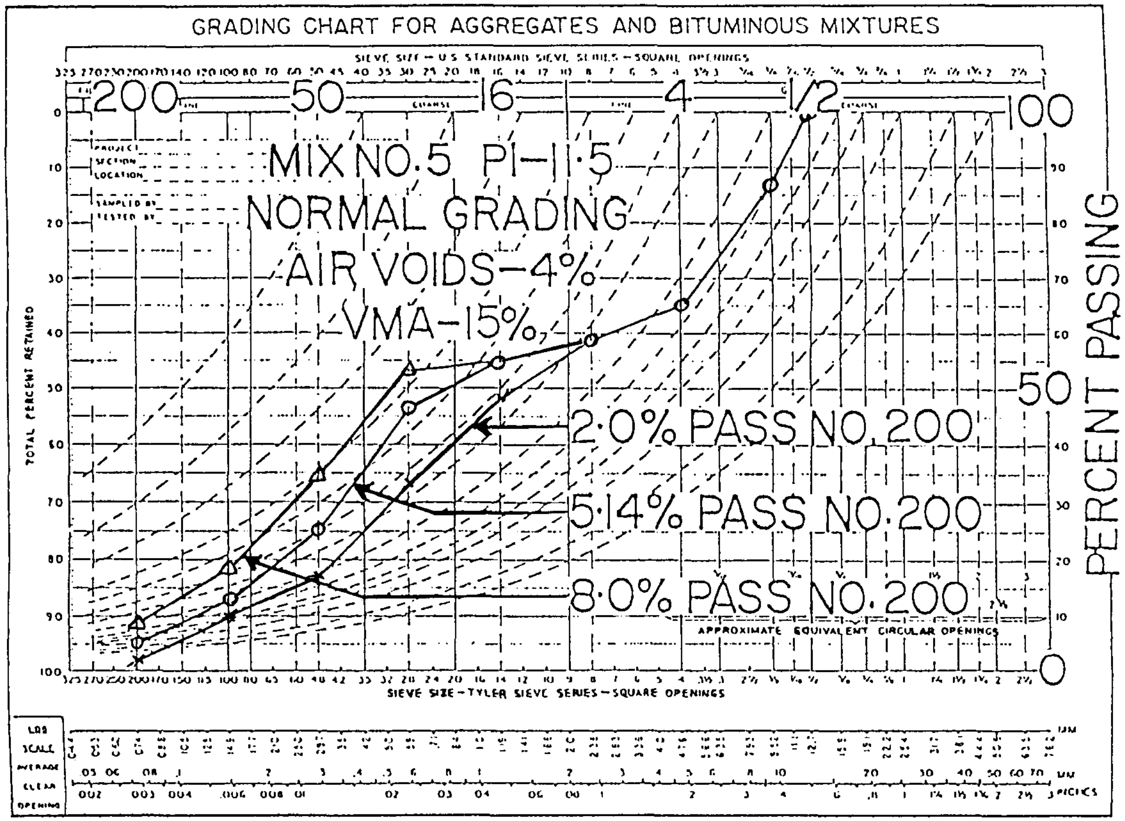

Figure 16 Grading curves illustrate reason for no increase in Marshall stability for an increase in percent passing No. 200.

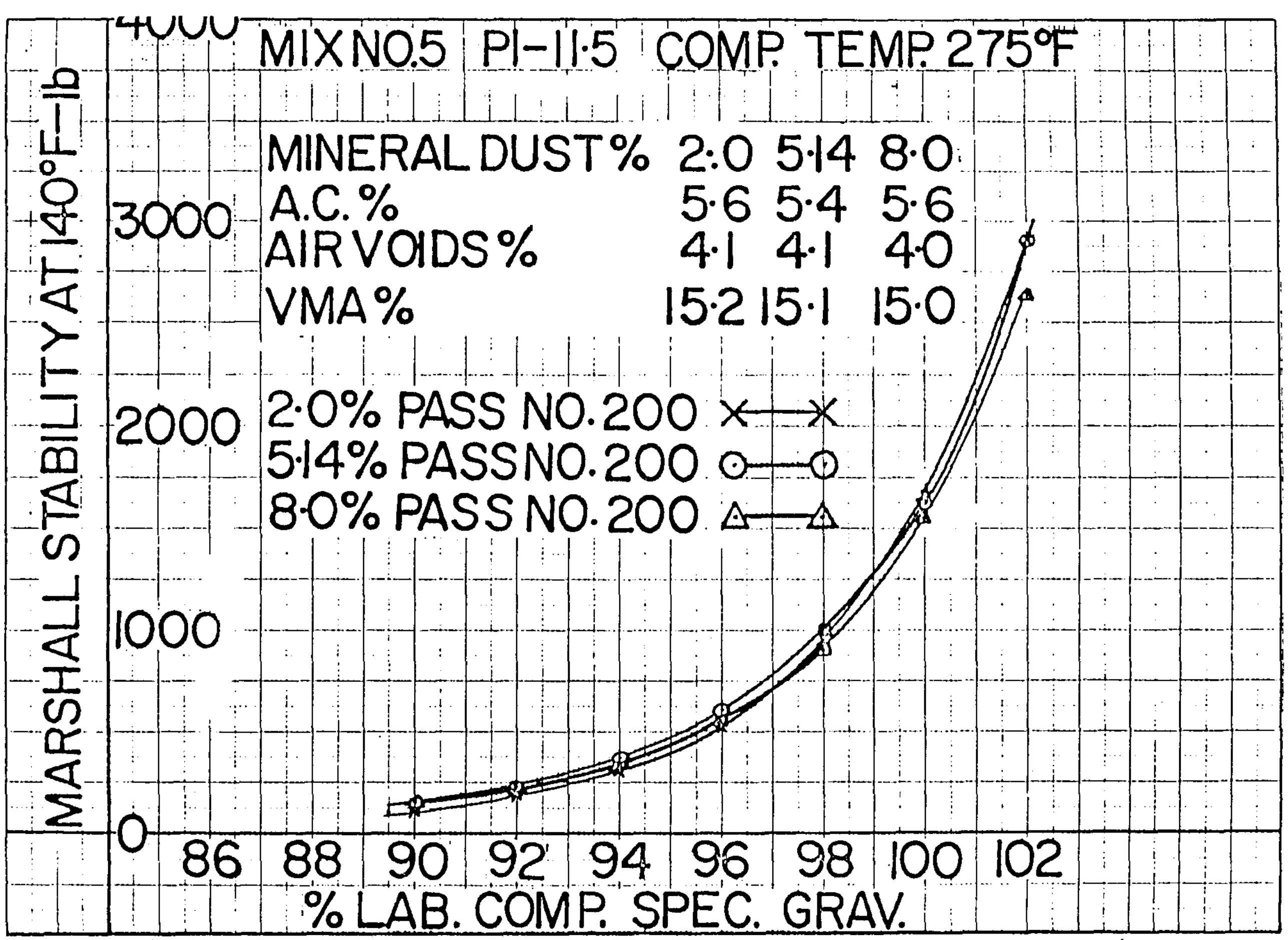

Figure 17 Illustrating no increase in Marshall stability for an increase in percent passing No. 200.

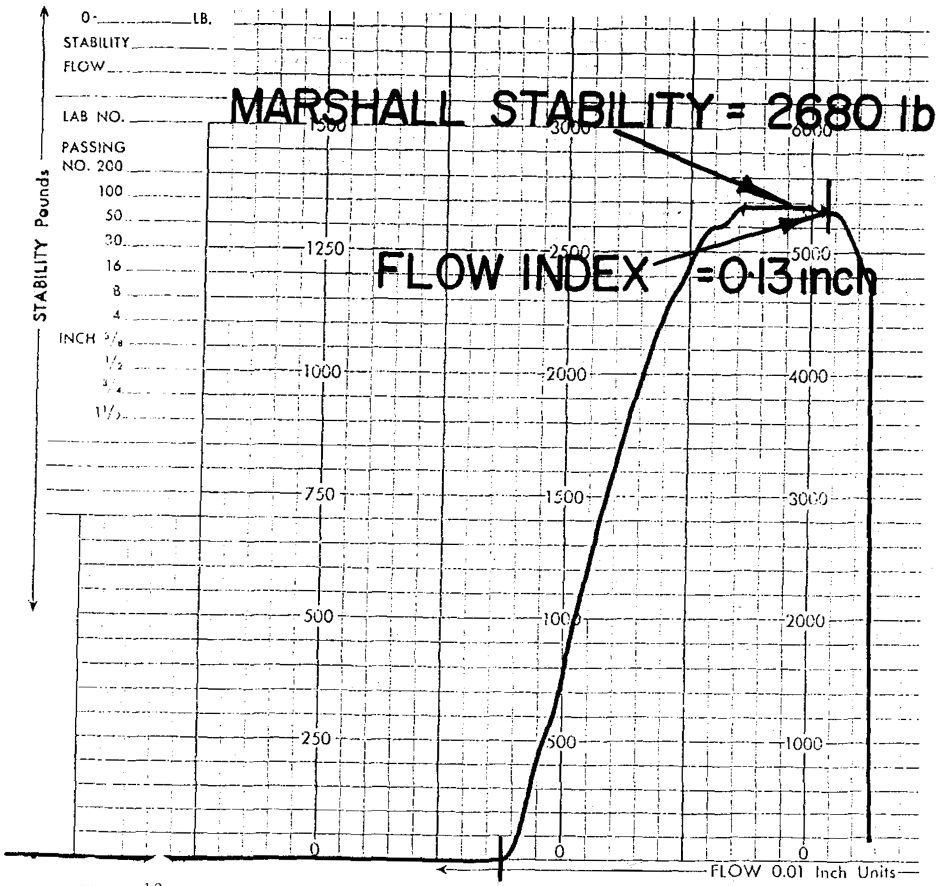

FIGURE 18 ILLUSTRATING FLOW INDEX FOR A WELL DESIGNED PAVING MIXTURE.

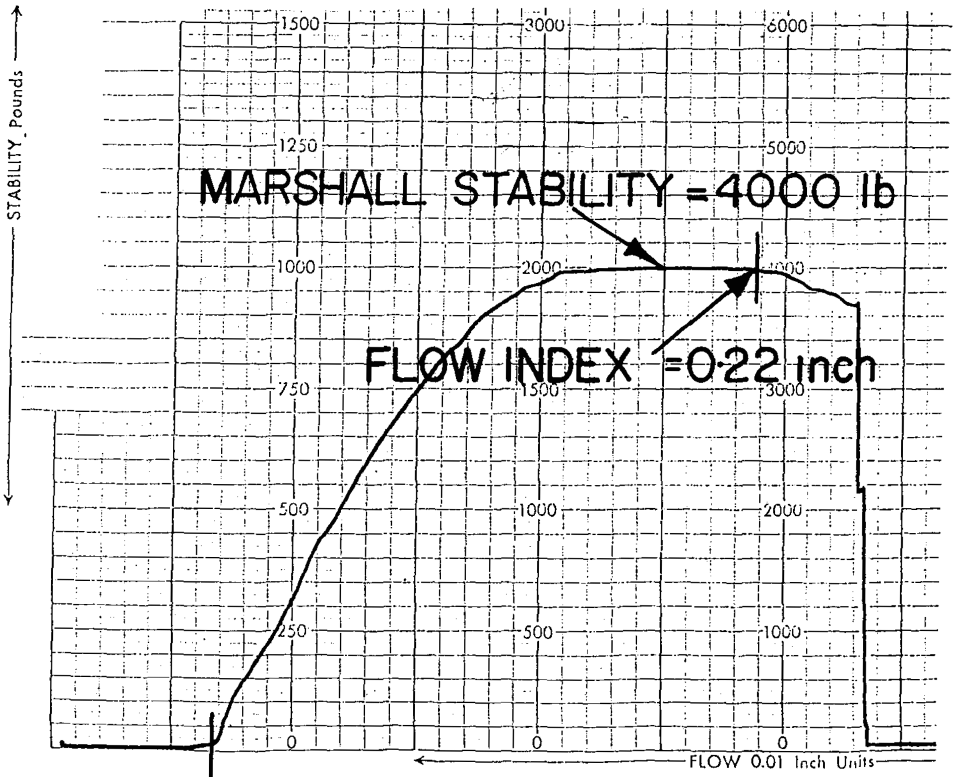

FIGURE 19 FLOW INDEX FOR AN OVER-ASPHALTED PAVING MIXTURE.

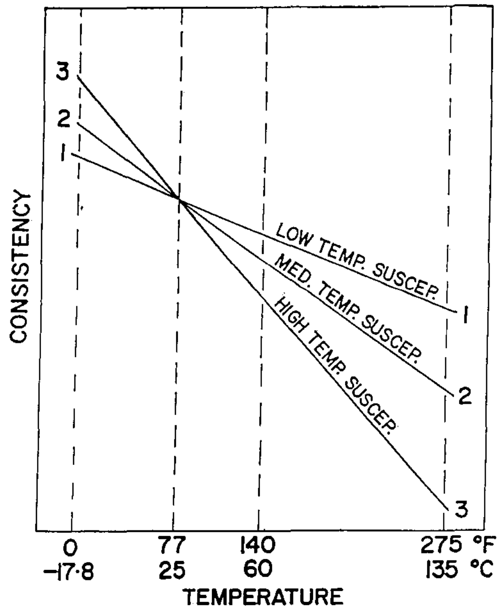

Fig. 20 SKETCH ILLUSTRATING DIFFERENT TEMPERATURE SUSCEPTIBILITIES OF PAVING ASPHALTS

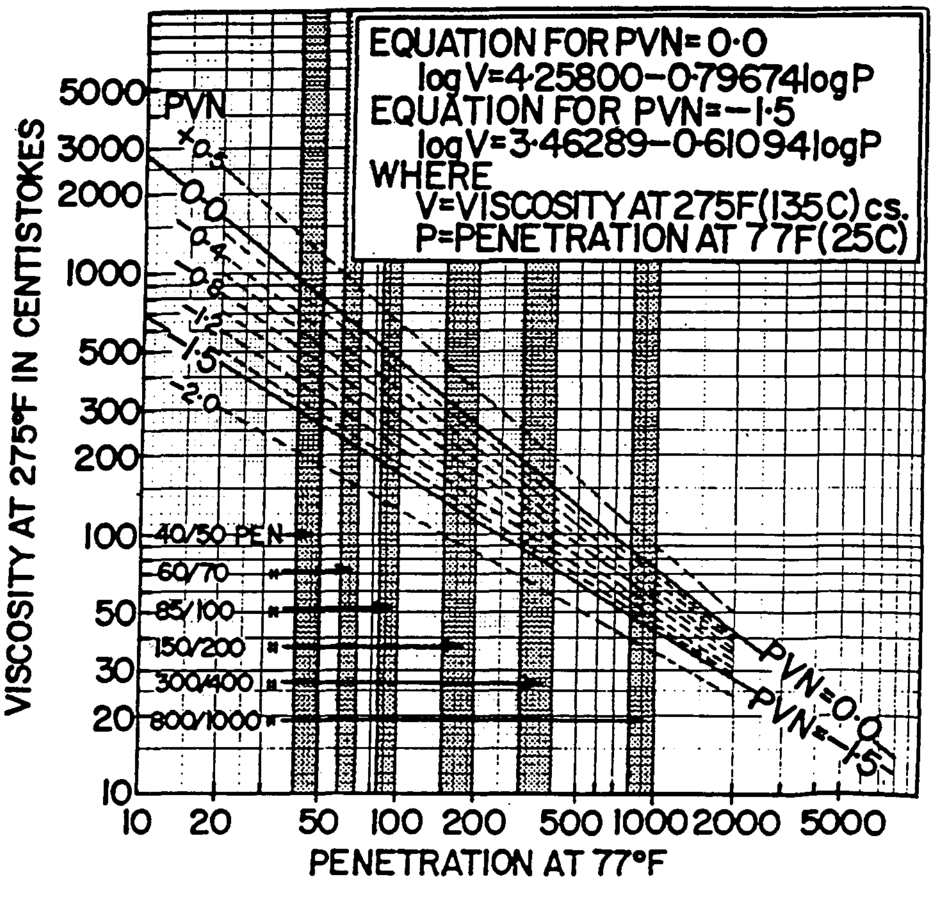

FIGURE 21 A CHART FOR THE DETERMINATION OF APPROXIMATE VALUES FOR PEN-VIS NUMBERS FOR ASPHALT CEMENTS.

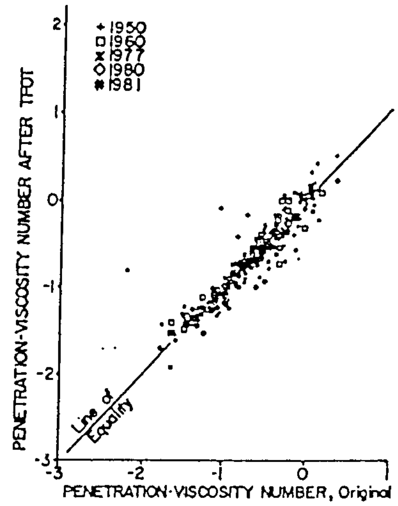

Fig. 22 Pen-Vis Number Before and After Thin-Film Oven Exposure.

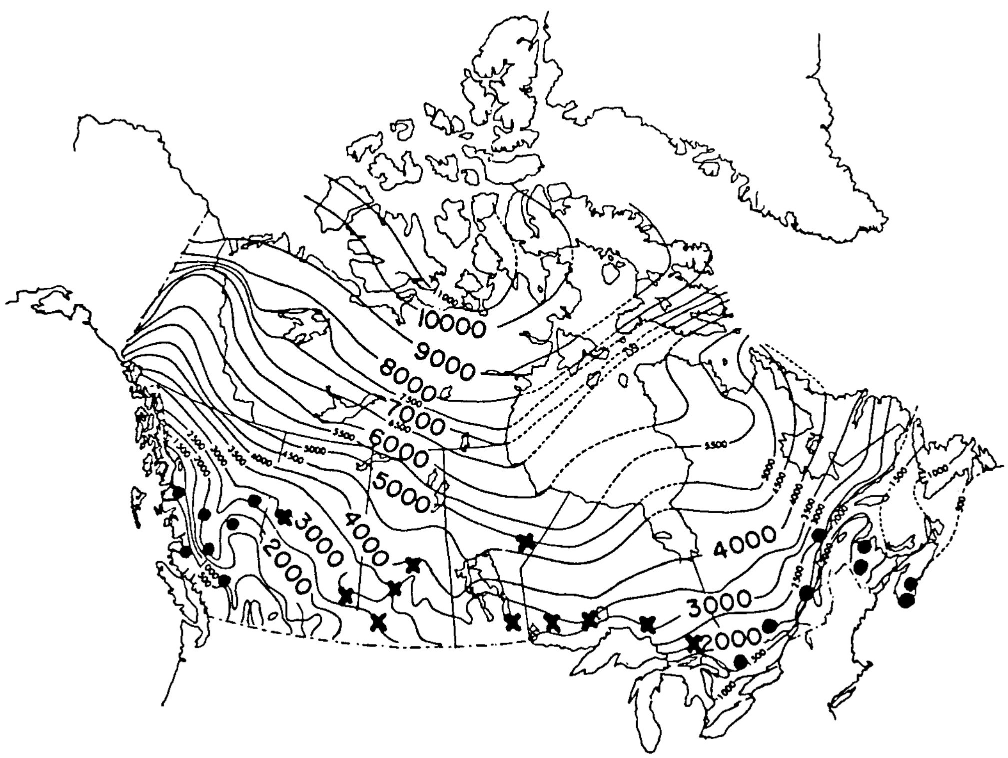

Figure 23 Approximate locations of the 26 airports (where x's indicate interior airports and o's indicate coastally associated airports). Freezing index countours are in °F days. (with credit to Ralph Haas)

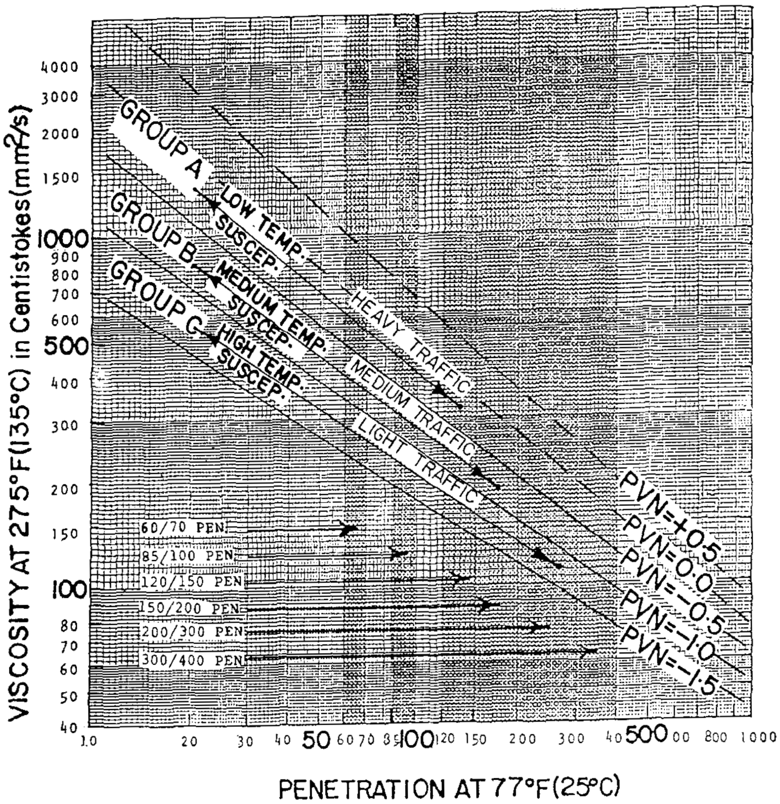

Figure 24 ILLUSTRATING A SPECIFICATION BASED ON PENETRATIONS AT 77°F (25°C), VISCOSITIES AT 275°F (135°C), AND TEMPERATURE SUSCEPTIBILITIES OF PAVING ASPHALTS.

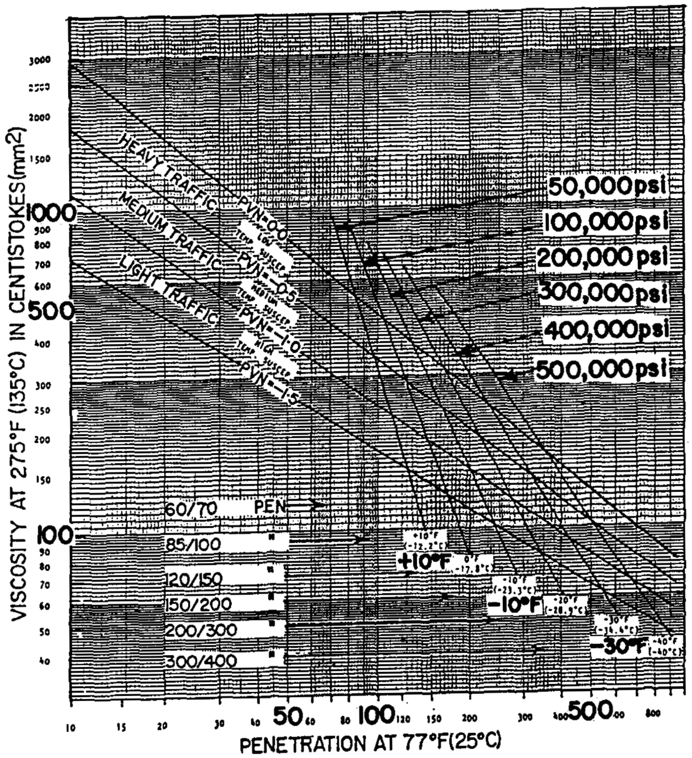

Fig. 25 CHART FOR SELECTING PAVING ASPHALTS WITH VARIOUS COMBINATIONS OF TEMPERATURE SUSCEPTIBILITIES AND PENETRATIONS AT 25°C TO AVOID LOW TEMPERATURE TRANSVERSE PAVEMENT CRACKING AT SELECTED MINIMUM WINTER PAVEMENT TEMPERATURES.

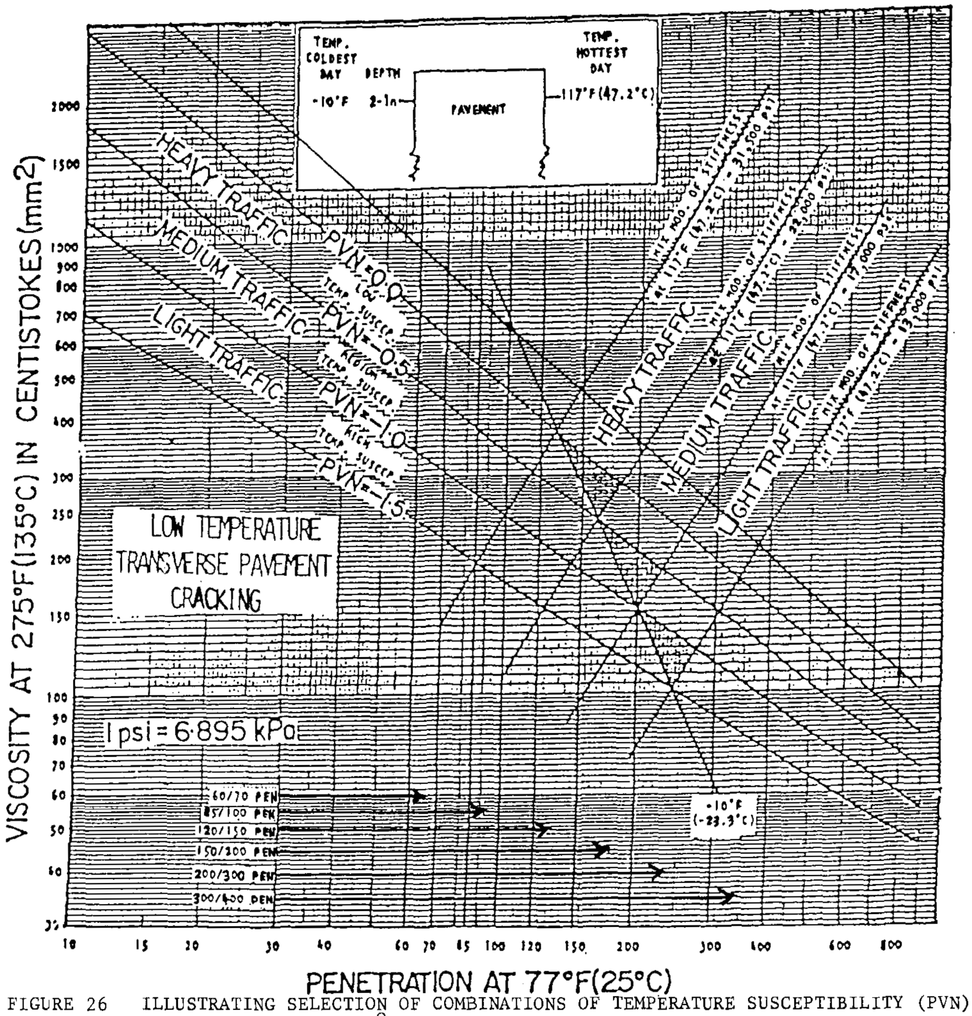

FIGURE 26 ILLUSTRATING SELECTION OF COMBINATIONS OF TEMPERATURE SUSCEPTIBILITY (PVN) AND PENETRATION AT 25^{o} C FOR PAVING ASPHALTS FOR HEAVY, MEDIUM AND LIGHT TRAFFIC IN COLD CLIMATES WHERE THE MINIMUM WINTER TEMPERATURE IS -0^{o}F (-23.5^{o}C).

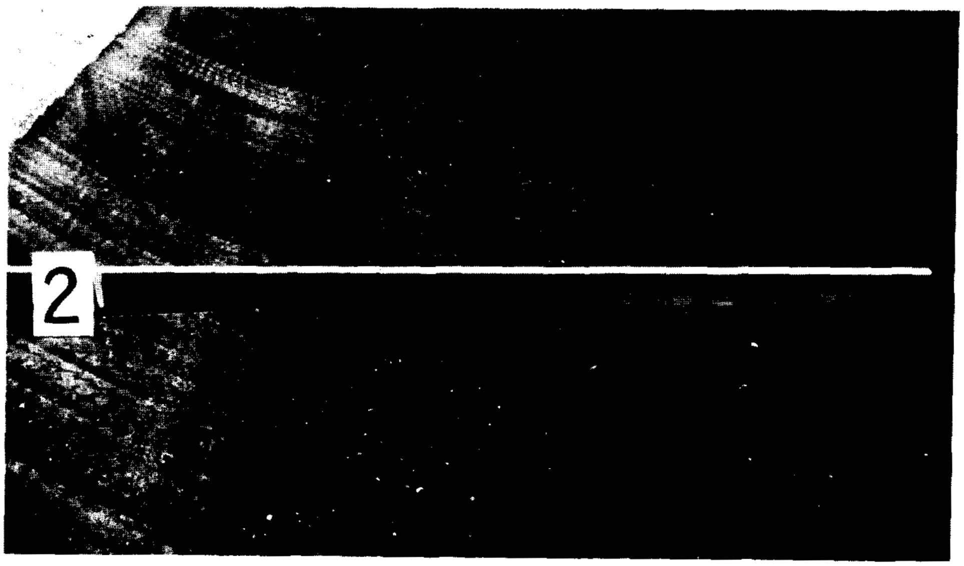

FIGURE 27 ILLUSTRATING ASPHALT PAVEMENT RUTTING

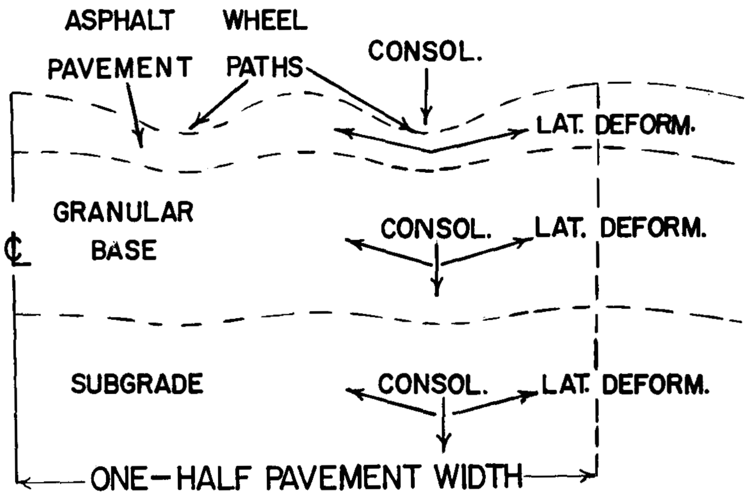

FIGURE 28 CAUSES OF PAVEMENT RUTTING

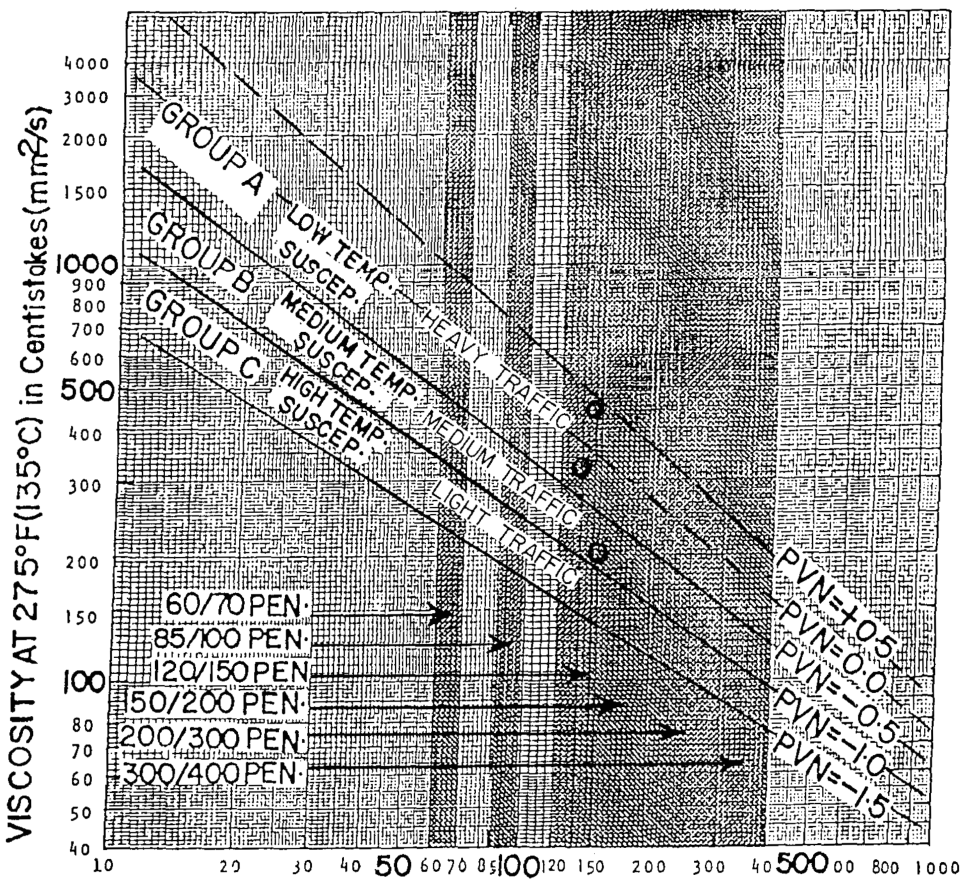

Figure 29 INFLUENCE OF A POLYMER ON THE TEMPERATURE SUSCEPTIBILITY OF A PAVING ASPHALT.

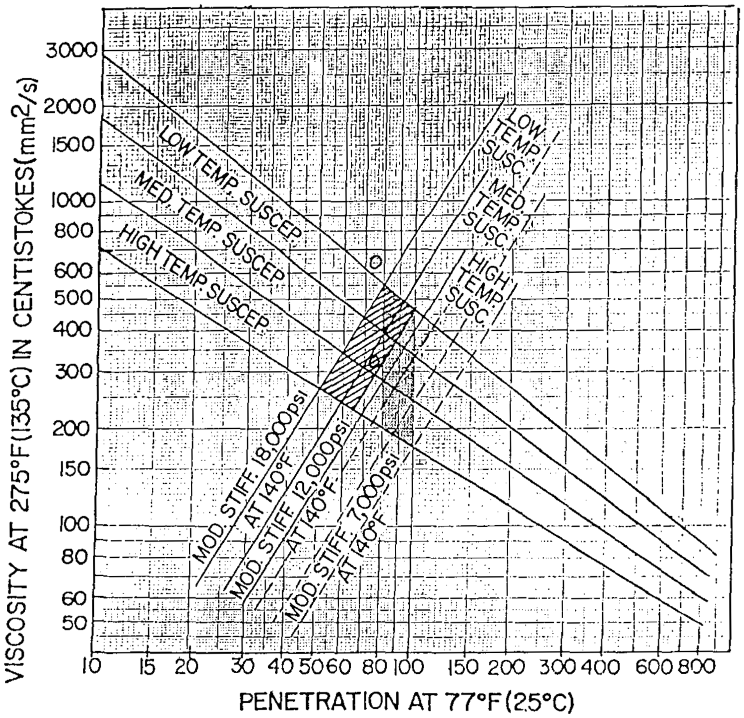

FIGURE 30 Illustrating how paving asphalt temperature susceptibility can be made to work for or against engineers in warm climates

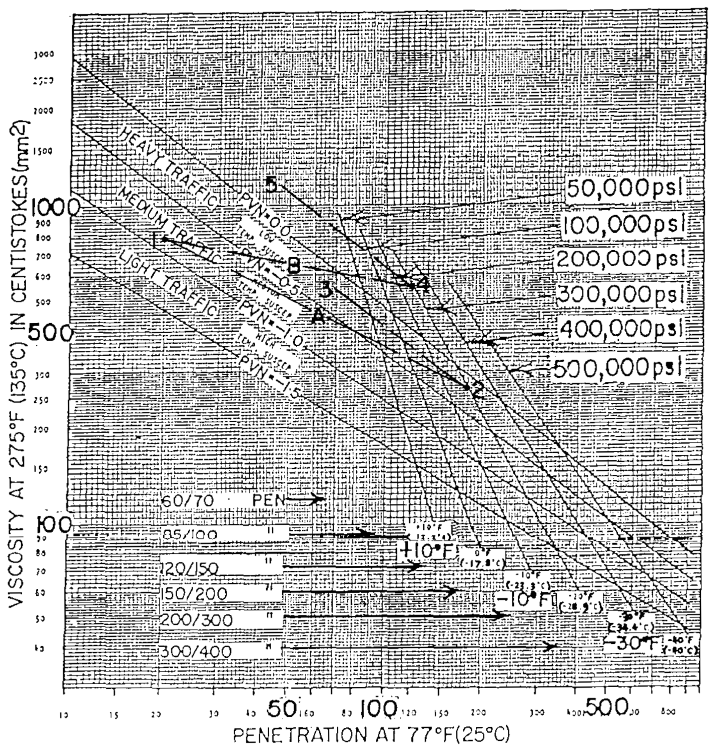

FIGURE 31 A guide for the design of recycled pavements in cold climates

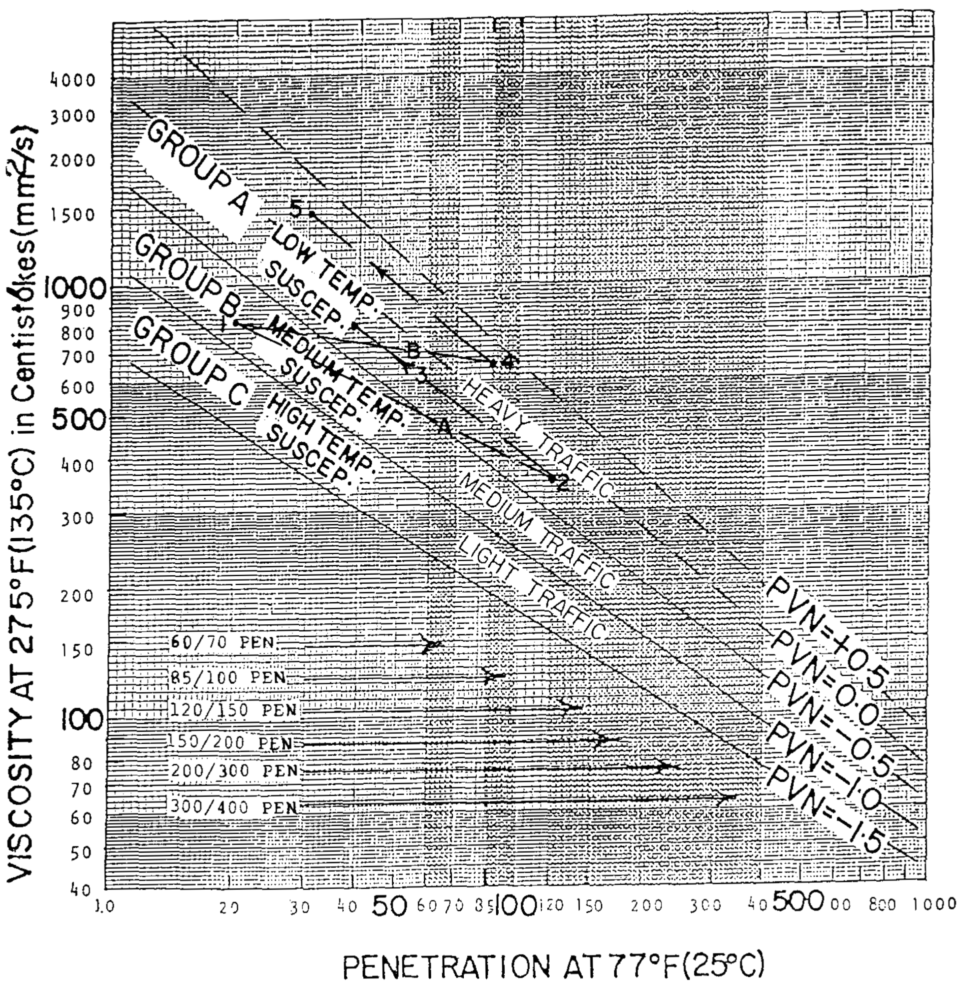

FIGURE 32 A GUIDE FOR THE DESIGN OF RECYCLED PAVEMENTS IN CLIMATES WITHOUT FROST.

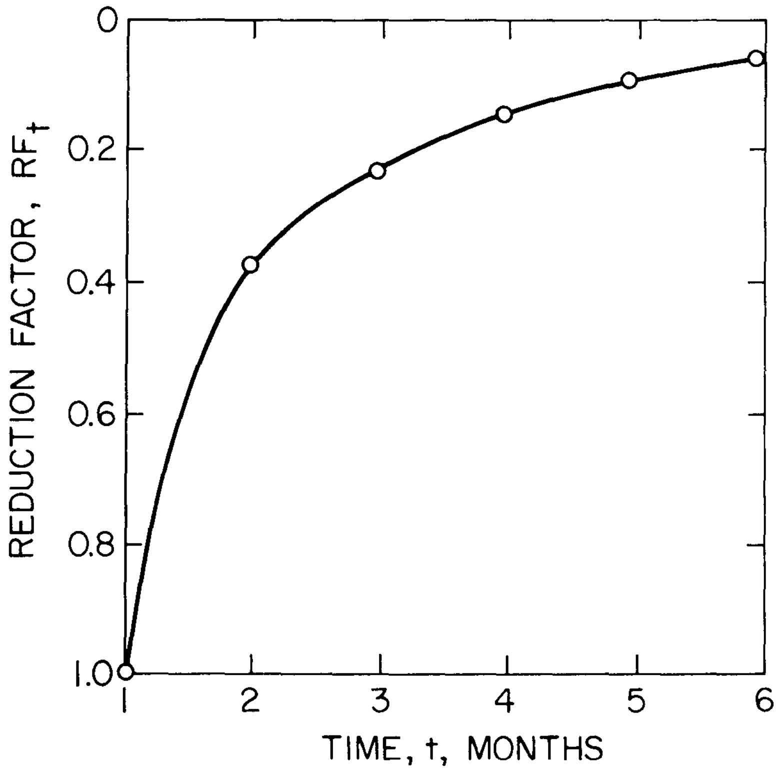

FIGURE 33 DEMONSTRATING THAT A CURING PERIOD OF ABOUT SIX MONTHS IS REQUIRED FOR AN ASPHALT EMULSION MIX TO DEVELOP ITS FULL STRENGTH.

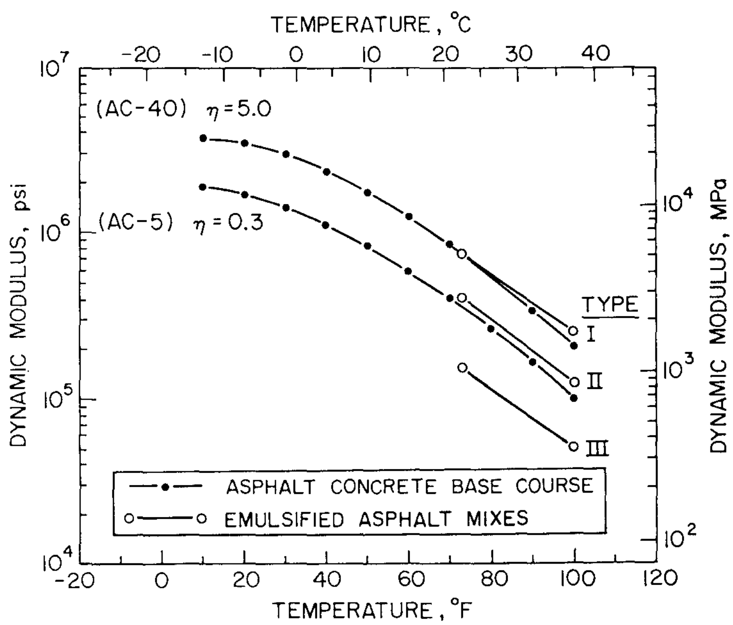

FIGURE 34 DEMONSTRATING THAT DEPENDING ON ITS TYPE (I, II, OR III) AN ASPHALT EMULSION MIX EVENTUALLY DEVELOPS STRENGTH EQUIVALENT TO THAT OF A CORRESPONDING HOT-MIX.