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A STUDY OF THE RELATIONSHIPS BETWEEN STRENGTH, DENSITY, PERMEABILITY, AND GRADATIONS OF AGGREGATE BASES

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

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SUMMARY

Accumulating evidence that inadequate subsurface drainage of some pavements was related to impervious base courses led to an investigation of the influence of low permeability fine materials on the physical characteristics of typical base courses. It was determined that a reduction in the minus 200 fraction of typical base courses from an average of 10% to an average of 7% was accompanied by a 1,000- to 10,000-fold increase in permeability with no adverse effects on California Bearing Ratio values or on maximum dry densities. These findings led to the development of a suggested revised gradation design range calling for from 6% to 8% minus 200 material as opposed to the old range of 8% to 12%.

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INTRODUCTION

It has long been known that excessive moisture in pavement foundations leads to excessive deflections, cracking of pavements, and the instability of subgrades. Also, it has been shown that wheel loads are many times more damaging on saturated and flooded sections than on dry pavements.[1] The deterioration of concrete pavement due to the "D" cracking phenomenon and the stripping of asphalt mixtures also can be partially attributed to excessive moisture in the pavement foundation.

The three main sources of water in the foundation are (1) high groundwater, (2) capillary rise, and (3) infiltration from the surface. High groundwater can be eliminated by constructing enough fill to raise the roadbed above the water table. By using a cohesionless material as a base and subbase material, capillary rise can be prevented. To eliminate the infiltration of water from the surface, there are two distinct alternatives. One is to cut off moisture to the roadbed by using an impervious membrane and properly maintaining ditches, shoulders, joints and cracks. However, it should be noted that experience has shown that it is impossible to entirely eliminate water from the subbase and subgrade.[1,2] The second alternative is to drain the water from the foundation as quickly as possible to prevent water related damage to the pavement. This is accomplished by the use of a base material of high enough permeability to remove infiltrating water in a short period of time.

It is well known that the permeability of a base material is governed largely by the percentage of fines in the aggregate; the permeability decreases with an increase in fines and vice versa.

Also, the amount of fines governs the strength and density of the subbase. Up to a certain percentage of fines the strength increases, and past that point the coarse aggregate tends to float in a matrix of fines and lose grain-to-grain contact to cause a reduction in strength. Similarly, the density peaks at a specific percentage of fines. Unfortunately, the optimum percentage of fines is not the same for the three mentioned properties.

The Virginia Department of Highways and Transportation uses densely graded aggregate bases designated No. 21, No. 21-A, or No. 22 for most of its pavement designs outside of the coastal plains. While aggregate designated No. 21-A is the most frequently used, all three types have job mix ranges of 8% to 12% for the minus 200 fraction. Specification tolerances are such that where the 12% job mix is used, a single sample of material with as much as 16% minus 200 material might be acceptable. The literature shows that base materials with such high minus 200 fractions are virtually impervious and provide practically no drainage.(2) Yet, design standards reflect the assumption that water will flow through the base material until it is intercepted by strategically placed cross drains and carried away from the pavement structure. On interstate pavements the base material typically extends through the shoulder and is "daylighted" at the point where the shoulder breaks downward toward the ditch. Clearly, as has been shown in numerous field investigations, the above two approaches to subsurface drainage design are totally ineffective where high percentages of minus 200 material are present in the base material. Furthermore, recent studies of interstate pavements showed dramatically better performance for pavements having bases with an average of 7% minus 200 as opposed to those with an average of 12% minus 200.(3)

While it is widely acknowledged that water enters pavement foundations through various points from surface infiltration, the quantity of water entering a given pavement and the rate at which this water is removed are not well known. Nor is it essential to know the exact quantities of water involved as long as the drainage characteristics of the pavement are defined. Johnson, in unpublished studies at the Research Council, found the coefficients of permeability listed in Table 1 for typical bituminous concrete mixtures used in the state.(4) All mixtures tested had been in service for at least one year so that densification under traffic had taken place. Perhaps the most meaningful data are those reported for full-depth pavement cores consisting of an S-5 surface mix, B-3 base mix, and any prime and tack coats used in contact with the bituminous layers. While it was noted that the results of permeability tests on the full-depth cores were somewhat variable, Johnson expressed the opinion that the prime and tack coats played a role in this variability and served to decrease the average permeability to the 1×10^{-4} cm/sec level. Clearly, to be effective in removing water from the pavement, the average permeability of the aggregate base layer must exceed that of the combined overlaying bituminous concrete layers.

Table 1

Coefficients of Permeability for Bituminous Concrete Mixtures

Mixture Type	Average Coefficients of Permeability (K), cm/sec.
S-5	2.3×10^{-4}
MS-5	5.8×10^{-4}
S-4	5.3×10^{-5}
I-3	3.4×10^{-4}
B-3	2.3×10^{-3}
Cores (S-5 & B-3)	1.0×10^{-4}

Recognizing that subsurface drainage problems existed, materials and research personnel in the Department discussed the possibility of modifying the specifications to provide a base material with improved drainage characteristics. It was, at the same time, recognized that the currently used aggregate bases are graded to expedite construction and enhance the structural capability of the pavement, and that large changes in gradation, particularly those changes that would detrimentally affect construction activities, could meet considerable resistance from construction personnel. Additional reluctance to changes can be anticipated from aggregate producers whose process yields a high level of fines so that excess materials would have to be discarded if coarser gradations were required. On the other hand, a cursory examination of the aggregate production on a statewide basis revealed that some producers find it necessary to add fine materials to their regular production in order to meet the existing specifications. Since these additional fines may be silts or silt clays they can be particularly detrimental to the permeability of the aggregate base.

OBJECTIVE AND SCOPE

The objective of this study was to investigate the present 21-A base material and determine if the maximum percentage of fines currently specified by the Department could be lowered to ensure that the base would have the desired permeability while maintaining its strength and density. It was recognized that the current base has a very low and possibly detrimental permeability, and believed that lowering the maximum percentage of fines allowed would correct the problem and thus increase the service life of roads.

The study attempted to represent all mixes of the 21-A subbase material in Virginia by sampling material from four districts throughout the state. In the Staunton, Culpeper, Lynchburg, and Richmond Districts a sample of stone was taken from a quarry recommended by the respective district materials engineers. The selected quarries produced 21-A material having approximately the maximum allowable percentage of fines according to the Department's present specifications.

In the following discussions these aggregate sources are designated as A, B, C, and D as identified in Table 2. Aggregate source E was added for special studies as described later in the report.

Table 2

Aggregate Sources

Designation	District	Supplier
A	Staunton	Mundy Quarry
B	Culpeper	Red Hill Quarry
C	Lynchburg	Blue Ridge Stone
D	Richmond	Chesterfield Quarry
E	Salem	Rockydale Quarry

EXPERIMENTAL APPROACH

In his book on pavement design, Yoder shows that the maximum density for a crushed stone occurs when the aggregate mix contains approximately 9% fines.[5] He also reports that the maximum California Bearing Ratio occurs when the mix contains 7% fines. It was noticed in examining his figures and graphs that when going from 9% to 7% fines there was little change in the compacted density of the stone. For these reasons, and in the hope of increasing the permeability, the present study determined the effects of lowering the percentage of fines in Virginia's 21-A base material to approximately 7%.

In Figure 1 the gradation curves representing the present 21-A (average of 10%) and the proposed percentages (average of 7%) of fines in base material are shown. The present curve was plotted using the centerline of the gradation given in the present specifications for 21-A material. The curve representing the changed gradation was plotted using Fuller's formula for maximum density.[5] In the formula

$$p = 100 \left(\frac{d}{D}\right)^{m},$$

where d represents the sieve in question; p is the percentage of material finer than the sieve by weight; D is the maximum size aggregate; and m is an exponential factor. For the purposes of this study the average percentages of fines desired (7%) was substituted in the expression for p; 0.074 (size of -200 material, in millimeters) for d; and 25.4 (maximum size aggregate contained in a No. 21-A mix, in millimeters) for D. From these values, the exponent m was calculated and used for determining the remaining portions of the gradation curve for 7% minus 200 material. In order to provide a gradation band for the study, tolerances used in the present specifications were applied to the 7% curve to get the 2% and 12% curves (see Figure 1). These curves were used when determining what percentages of each size aggregate to use when mixing the subbase material to be tested. Table 3 gives the current and the experimental gradations. It should be noted that suitable tolerances are provided by the specifications once the producer chooses his job mix from the current design range.[6]

Table 3

Current Design Ranges for 21-A Base Material

Sieve Size	Percentage by Weight Finer than Each Sieve	Experimental Gradations
2"	100	100
1"	94 - 100	90 - 100
3/8"	63 - 72	47 - 81
No. 10	32 - 41	19 - 43
No. 40	16 - 24	6 - 24
No. 200	8 - 12	2 - 12

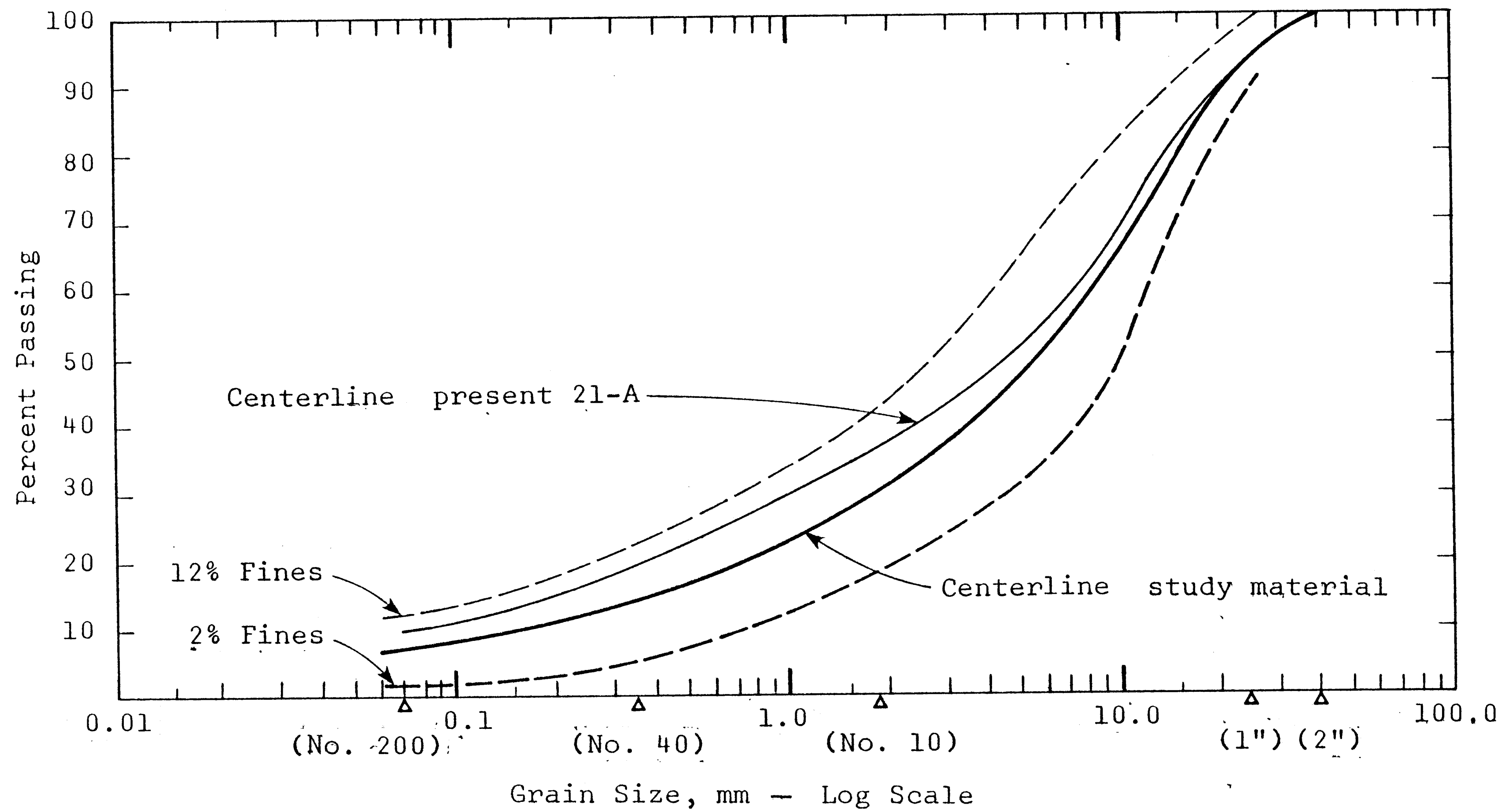

Figure 1. Gradation curves for various percentages of fines.

When a stone sample was received from a quarry, it was dry sieved and remixed according to the amount needed for the percentages given in Figure 1. The present gradation mix of 10% fines was tested, as was a mix of 7% fines along with mixes representing the upper and lower extremes of 2% and 12% fines. Testing the 7% mix along with its extremes indicated the effect to be produced by lowering the optimum percentage of fines to 7%.

After sieving, the maximum density, optimum moisture, CBR value and permeability of each mix (2%, 7%, 10%, and 12% fines) were determined for each stone sample. All tests were run according to AASHTO specifications. The maximum density and optimum moisture content were determined from a Proctor density curve obtained according to AASHTO T99-70 (Method C). The strength of the subbase material was determined by calculating its CBR value in accordance with AASHTO T193-63. A sample of stone was mixed and brought to optimum moisture, then compacted to maximum density in a standard CBR mold. After the specimen had soaked for four days, its CBR value was obtained. The permeability of the subbase material was determined in accordance with AASHTO T215-70. A sample was compacted to maximum density in a 4-inch diameter mold to produce a sample approximately 3 inches in length. The sample was soaked to ensure saturation, then the permeability was determined by running a constant head or a falling head test, whichever was appropriate.

RESULTS AND DISCUSSION

Permeability Test Results

The results of the permeability tests conducted on aggregate base courses are shown in Figures 2 and 3. For each source of materials, one from each of five districts, the permeability is plotted as a function of the minus 200 material in the samples. For clarification, Figure 3 is a composite graph showing the average permeabilities from all sources combined as a function of the minus 200 fraction in the base course. Superimposed on this figure are points indicating the average permeabilities of S-5 and B-3 bituminous concretes, including a point for cores representing the combined thickness of the S-5 and B-3 courses. Figures 2 and 3 both show permeability values on the order of 1 x 10^{-2} cm/sec for minus 200 fractions up to 7%, while there is a sharp drop in the permeabilities somewhere above that value. The presently specified aggregate base with an average of 10% minus 200 has an average permeability of approximately 1 x 10^{-5} cm/sec. It may be noted in Figures 2 and 3 that only the Salem District source (E) was tested at a minus 200 fraction of 8.5%. These tests were conducted after

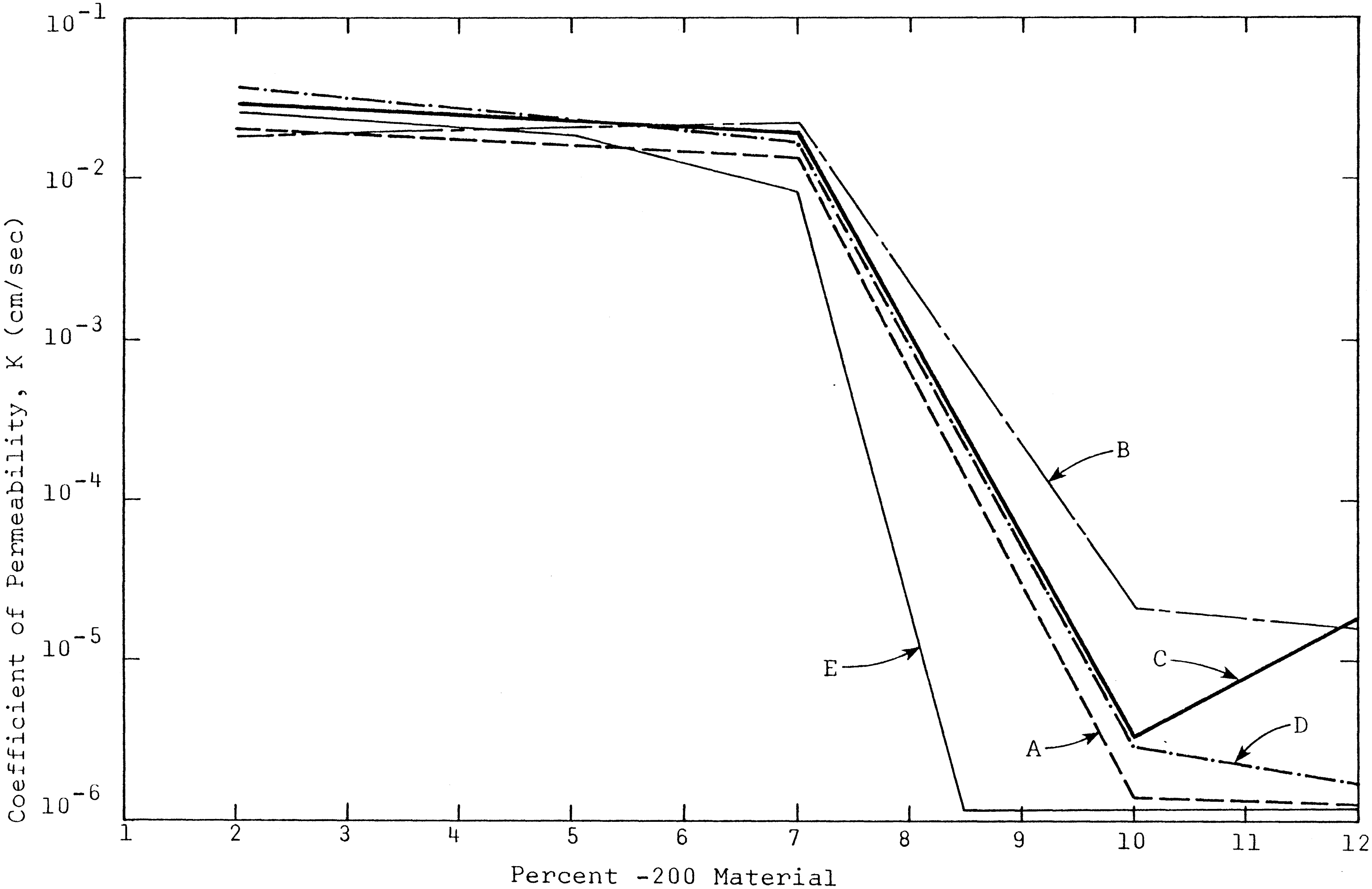

Figure 2. Coefficient of permeability vs. percentages of -200 material.

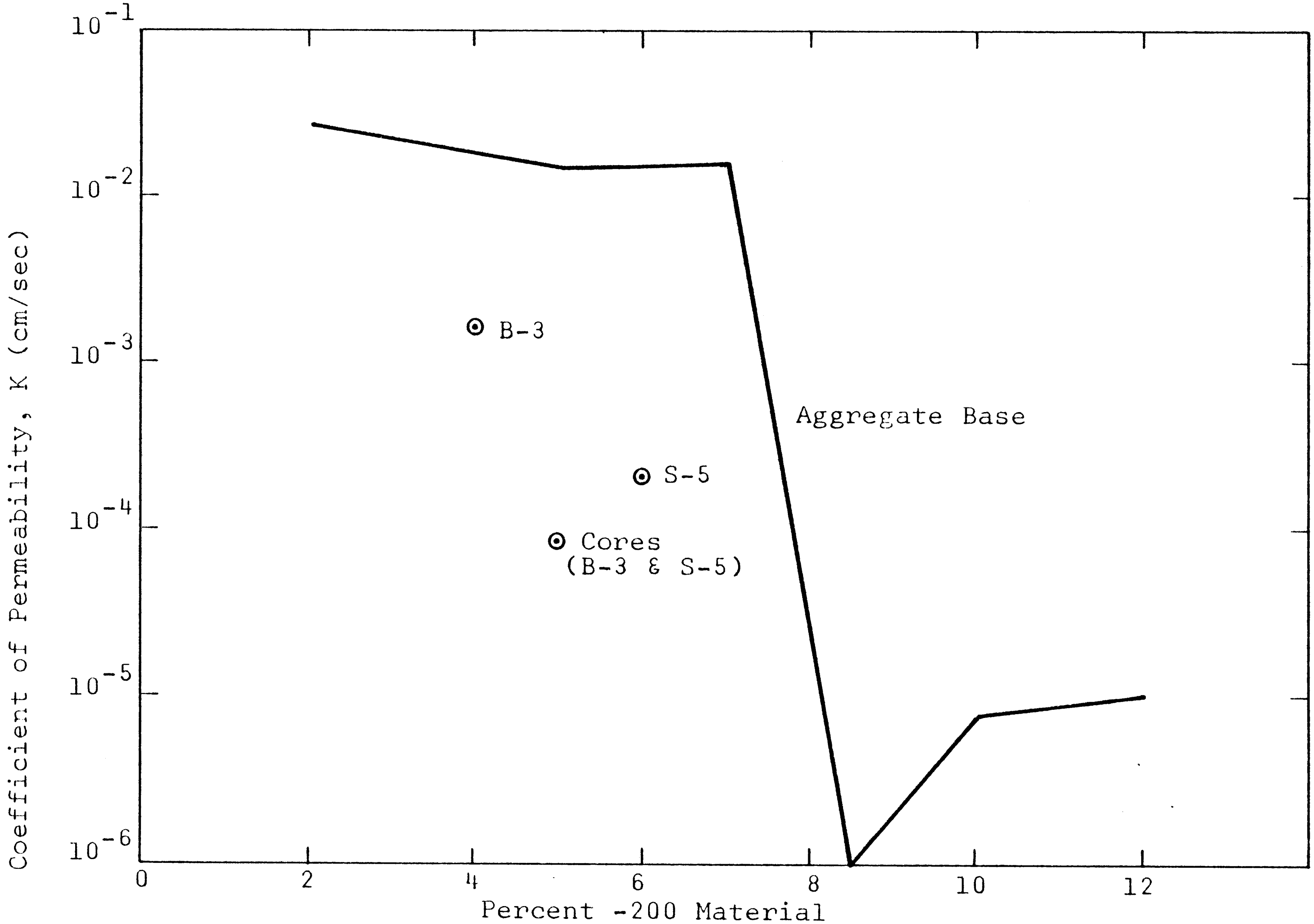

Figure 3. Coefficient of permeability vs. percentages of -200 material. Composite for all sources of materials.

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those on the other four sources had been completed and in an effort to more precisely define the minus 200 fraction at which the sharp decline in permeability occurs.

The implications of the sharp reduction in permeability with increased minus 200 material can be perceived better through a rough analysis of pavement infiltration rates as opposed to drainage rates. The familiar flow equation (Darcy's law) is expressed as

$$Q = KiA,$$

where

Q = flow rate cc/sec.,

i = hydraulic gradient cm/cm, and

A = area of flow.

The infiltration rate (Q_i) through a unit area of an average bituminous concrete pavement ($K = 1 \times 10^{-4}$ cm/sec) under a water film thickness of 1 cm may be expressed as

$$Q_i = 1 \times 10^{-4} \times 1 \times 1 = 1 \times 10^{-4} \text{ cc/sec.}$$

The outflow rate (Q_o) through a unit area of a typical aggregate base built to a normal cross slope of 2% may be expressed as

$$Q_o = 0.02 \times 1 \times K = 0.02K.$$

Ideally, the drainage characteristics of the total pavement should provide at least as much outflow as infiltration during a rainstorm; i.e.

$$\frac{Q_i}{Q_o} \leq 1.$$

As shown in Table 4, this criterion is satisfied when the minus 200 fraction of aggregate bases is less than 7%. In this case the drainage rate is approximately twice the infiltration rate ($Q_i/Q_o = 0.50$). In the case of minus 200 fractions in excess of 8.5% the aggregate base coefficient of permeability is approximately 1×10^{-5} cm/sec and $Q_i/Q_o = 500$; i.e. the water-carrying capability of the bituminous concrete layers is 500 times that of the aggregate base.

Table 4

Ratios of Infiltration Rates to Drainage Rates

Minus 200 (%)	K_o	Q_i/Q_o
$\leq$ 7.0	1×10^{-2}	0.50
$\geq$ 8.5	1×10^{-5}	500

While this analysis does not identify the exact point at which there is a sharp decrease in permeability due to increased minus 200 material, it is evident that the point lies somewhere between 7% and 8.5% minus 200. The point at which the sharp drop occurs can be interpreted as that point at which all the interstices between coarse aggregate particles have been filled with fine materials and the passage of water is dramatically reduced. For minus 200 fractions beyond that point it is likely that the coarse aggregate particles are suspended in a matrix of fines and no further reduction in permeability occurs.

Density Study

In Figure 4 the maximum dry densities for each of the aggregate sources studied are plotted as functions of the percentage of minus 200 material. In the graph it can be noted that the maximum dry densities for sources A, B, C, and D occurred somewhere between the 7% and the 10% minus 200 points. Again, source E was tested later in an effort to better define the percentage of minus 200 corresponding to the highest maximum dry density. As the shapes of all curves show, the highest maximum is at approximately 8.5% minus 200.

Perhaps the most significant finding from this part of the study is that there is little difference between the maximum density obtained for 10% minus 200 (the median of the present 21-A design range) and 7% minus 200 (the median of the experimental gradation range). In all cases there was a small decrease in the maximum density attributable to the lower percentage of minus 200. The largest difference was a 2.5 lb./cu.ft., or approximately 1.8%, reduction in density when fines were reduced from 10% to 7%.

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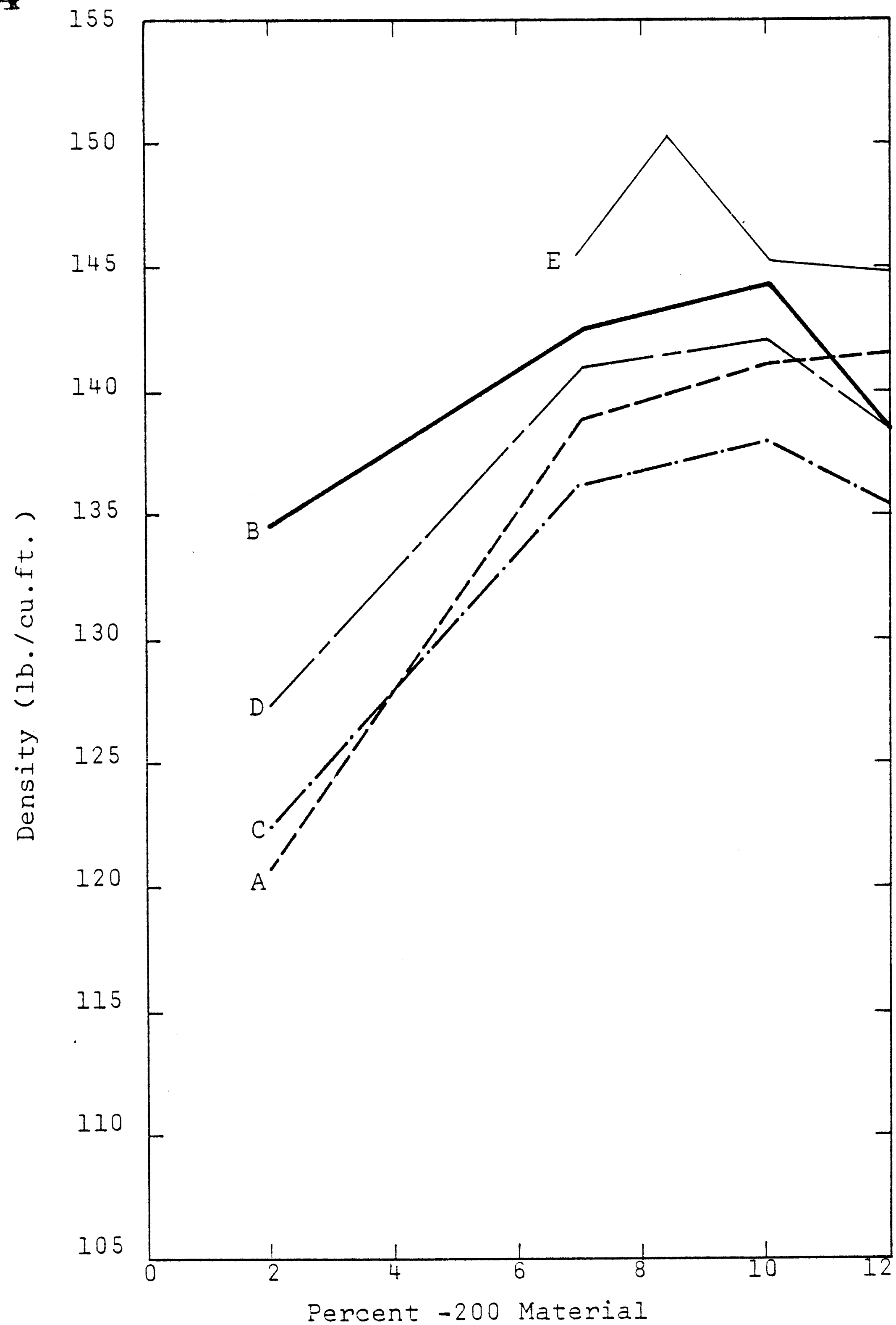

Figure 4. Maximum dry density vs. percentage of -200 material.

In comparing densities corresponding to 7% fines to those corresponding to 12% fines it is evident that the former tend to be slightly higher. On the lower side of the fines range, densities corresponding to 2% fines are considerably less, in some cases as much as 18 lb./cu.ft., than those for 7% or 12%. Such low maximum densities corresponding to such a low percentage of fines are considered unlikely to occur often in practice due to the fact that most aggregate producers in the state tend to have somewhat more than 2% fines in their normal production. In addition, when small percentages of fines were present it was observed in the laboratory compaction work that coarse aggregate particles tend to abrade and produce additional fines. Such behavior might or might not occur under field compaction conditions.

CBR Studies

It would logically be assumed that since going from 10% to 7% fines causes a small reduction in the density of the base course, there would be a corresponding decrease in strength. Figure 5, where the results of CBR tests are plotted, shows that this reduction does not occur and that, in fact, the maximum CBR occurs at around 5% to 7% fines. Base materials having CBR's of from 20% to 50% are typically considered as good while those with CBR's in excess of 50% are considered excellent. In four of the five cases studied the CBR's exceeded 50% for 7% or less fines. In all cases there was a significant drop in the CBR values for 10% and 12% fines. Once again, the previously outlined reasoning applies; i.e. for the higher percentage of fines the coarse aggregate particles are suspended in a matrix of fines such that they do not come in contact and strength is adversely affected.

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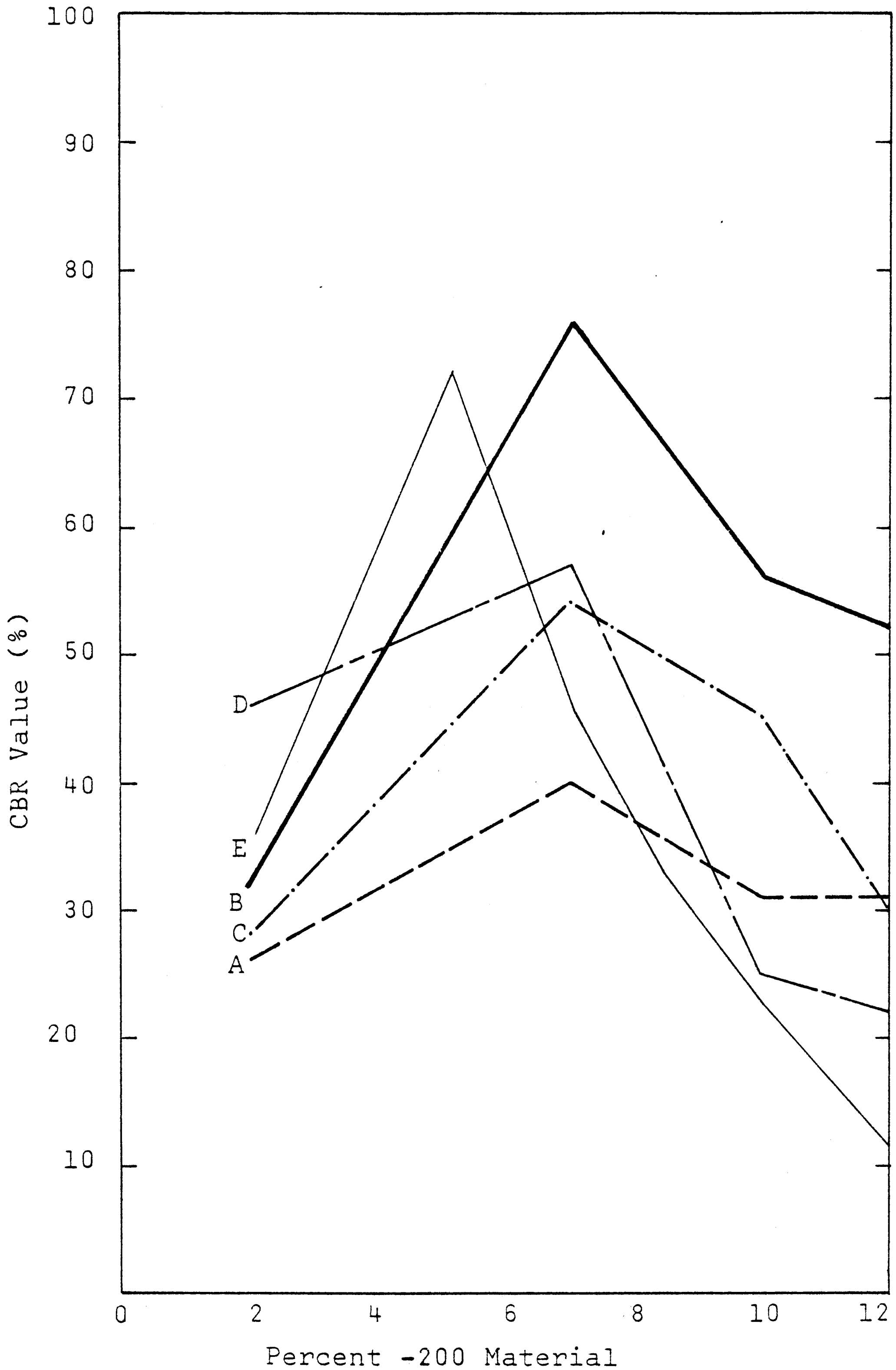

Figure 5. CBR value vs. percent of -200 material.

RELATED PRACTICES BY OTHER AGENCIES

Many highway departments in other states and various other agencies have found results similar to these presented in this report. In separate tests run by the Corps of Engineers and the University of Maryland, the permeabilities for densely graded base coarse materials containing 0%, 5%, and 10% passing the 200 sieve were found to be 10^{-4}, 10^{-5}, and 10^{-6} cm/sec., respectively.(7) In a cooperative study between the Ohio Department of Transportation and the Federal Highway Administration it was found that Ohio's subbase material No. 310 (old specification I-22) had permeabilities ranging from 10^{-3} cm/sec. and higher for 7% or less of fines. Above this percentage, the permeability decreased to as low as 10^{-6} cm/sec. In their study, samples were taken from roads at different locations in Ohio. The permeability was found, the pavement condition noted, and the density determined. The similar densities found for Virginia's 21-A material, along with the fact that Virginia's 21-A specifications are within the specifications for Ohio's 310 subbase material, indicate that similar subbase materials are used in the two states. Out of seven Ohio locations for which samples showed permeabilities of 10^{-3} cm/sec. or better, it was reported that in six cases the road condition was "good." In three out of the four cases where the coefficient of permeability was 10^{-4} and below, the road was reported to be in either "fair to poor" or simply "poor" condition. Ohio also did a study on the variation of the permeability of an aggregate base layer with various percentages of fines. They found permeabilities of 10^{-2}, 10^{-3}, and 10^{-5} cm/sec. for 0%, 2.5%, and 5.0% fines, respectively. Although these values do not correspond to those obtained in Virginia, they clearly show how the percentage of fines affects the permeability of a base course material. In their concluding remarks it was stated that:

> The results of field and laboratory evaluation of subbase material 310 indicated that the drainage material exhibited a wide range of performance characteristics. It was noted, in general, in all pavements exhibiting satisfactory performance, the subbase material 310 was very clean (with percent fines of 5% to 10%) and the drainage connector material was also void of any clay lumps or dirt. The coefficient of permeabilities of such material ranged from 10^{-3} to 10^{-2} cm/sec.
>
> Those subbase materials 310 having higher percentages of fines (10% to 15%) were, in almost all cases, associated with poor pavement performance.

Other states surrounding Virginia have also found similar results as reflected in their specifications. Maryland specifications call for the percentage of fines to be 0% - 10% ± 3% for densely graded aggregate base material.[8] Maryland engineers have reported that even at this percentage their base materials were considered impermeable with permeabilities ranging from 10^{-6} to 10^{-4} cm/sec. For this reason, they incorporate other means of drainage during construction. North Carolina specifications limit the percentage of -200 material to 12% for the coastal-piedmont section and 10% in the mountains; the specifications call for 8% ± 4% and 7% ± 3%, respectively. F. T. Wagner, head of the Materials and Test Unit of North Carolina, is of the opinion that the maximum of 15% fines used in Virginia is too high and that 12% is a more realistic maximum.[9] Kentucky specifications call for 5% to 12% passing the No. 200 sieve but, according to the director of the Division of Materials, the percentage of -200 material should not exceed 10%. He believes that a value of 4% to 6% would be more suitable because of its drainage capabilities, assuming the base course is thick enough to ensure adequate strength.[10] And last, the National Crushed Stone Association's Guide Specification recommends that after compaction a maximum of 10% passing the No. 200 sieve be allowed for a subbase material and even less than this would be preferred.[11] Therefore, evidence from this study and from various other studies indicates that an average of 7% to 8% fines with a maximum of 10% to 12% should be allowed for crushed aggregate base materials.

CONCLUSIONS

The results discussed above have led to several conclusions having a bearing on the performance characteristics of crushed aggregate base materials used by the Department. These are summarized below.

1. To ensure aggregate bases that have reasonable drainage capability it is necessary to restrict the minus 200 fraction to approximately 7%, on average.

2. If 7% minus 200 is provided there will be a small reduction in the maximum dry density and there may be a significant increase in CBR values.

3. Minus 200 fractions much below 7% give no significant improvement in permeability, while both the maximum density and CBR values can be detrimentally affected.

4. To assure aggregate bases having the most desirable properties it will be necessary to fairly rigidly control the minus 200 fraction at approximately an average of 7%.

PROPOSED REVISION TO SPECIFICATION

In view of the conclusions enumerated above it is evident that consideration should be given to a revised aggregate base gradation taking into account the benefits of reducing the allowable percentage of minus 200 material. The proposed revision is given in Table 5, where both the centerline gradation and the design range are listed. The centerline gradation given is the one used for the study purposes discussed earlier. The design range chosen provides aggregate producers latitude in chosing a job mix formula, and is similar to that provided in Table II-6 Section 209.02 of the 1978 Road and Bridge Specifications, except for the No. 200 sieve fraction.(6) In the case of the No. 200 sieve, it is believed, for reasons given earlier, that the 6% to 8% design range is as much latitude as can be provided without detrimentally affecting the properties of the aggregate base.

Table 5

Proposed Revised Gradations for Crushed Aggregate Base

Sieve Size	Percentage by Weight Finer than	
	Centerline Gradation	Design Range
2"	100	100
1"	95	90 - 100
3/8"	64	59 - 69
No. 10	31	26 - 36
No. 40	15	11 - 19
No. 200	7	6 - 8

Since there is no reason to believe that the gradation changes proposed would affect process variability to an appreciable degree, the process tolerances given in Table II-8, Section 209.02 of the 1978 Road and Bridge Specifications are recommended for the revised gradation.(6) These process tolerances were shown by Runkle to be realistic for aggregate base materials No. 21, 21-A, and 22.(12)

It may be noted that the process tolerances on individual samples for the No. 200 sieve are ± 4%. Therefore, a producer who choses a job mix of 8% minus 200 (i.e. on average he will produce 8% minus 200) will normally have a range of 4% to 12% minus 200. Then, if the assumption is made that the process is such that minus 200 values are normally distributed, there should be sufficient open graded material intertwined with the relatively impervious material that adequate drainage would result.

The price adjustment system given in Section 209.09 of the 1978 Road and Bridge Specifications is sufficiently rigorous on the No. 200 sieve to provide economic incentive to meet the revised specifications.

Construction Considerations of Revised Specifications

The most often heard objection to utilizing base materials graded so as to provide adequate drainage is in the purported difficulty of construction. It is argued that reducing the fines produces a harsh, difficult to compact material that makes density specifications difficult to meet. It is the opinion of the authors that this argument will not be applicable to the proposed revised aggregate base material. Two reasons for this opinion are:

1. The entire gradation is being changed rather than just the fines fraction. Therefore, no gap grading is anticipated, and, as the tests have shown, the maximum dry densities are not seriously affected.

2. Density is now accepted on the basis of the control strip approach under which the contractor, on the road, establishes his own level of compactive effort relative to the material he is using.

The above reasons notwithstanding, the first pavement constructed under the revised specifications should be closely studied to identify any difficulties in construction that might develop.

RECOMMENDATIONS

The revised crushed aggregate base gradation set forth in the previous section of this report is recommended for consideration by administrators of the Department. It is further recommended that the revised gradation be applied to at least one construction project at the earliest practicable date and that the project be studied to identify any difficulties with using the revised material and to verify the assumed variabilities in the materials produced.

REFERENCES

1. Majidzadeh, K., "Evaluation of Pavement Subsurface Drainage Conditions in Ohio," Ohio Department of Transportation and Federal Highway Administration, U. S. Department of Transportation, December 1976.

2. Cedergren, H. R., "Development of Guidelines for the Design of Subsurface Drainage Systems for Highway Pavement Structural Sections," prepared for the Office of Research, Federal Highway Administration, Washington, D. C., Report No. FHWA-RD-73-14, February 1973.

3. McGhee, K. H., "A Review of Pavement Performance on Virginia's Interstate System," Virginia Highway & Transportation Research Council, November 1976.

4. Johnson, C. G., "Permeability of Bituminous Concrete," unpublished Report, Virginia Highway & Transportation Research Council, 1977.

5. Yoder, E. J., and M. W. Witczak, Principles of Pavement Design, 2nd ed., John Wiley & Sons, 1975.

6. Virginia Department of Highways and Transportation, Road and Bridge Specifications, 1978.

7. Moynahan, T. J., Jr., and Y. M. Sternberg, "Effects on Highway Subdrainage of Gradation and Direction of Flow Within a Densely Graded Base Course Material," Transportation Research Record 497, Transportation Research Board, Washington, D. C., 1974.

8. Memorandum from W. B. Greene, chief, Bureau of Soils and Foundations, Maryland Department of Transportation, State Highway Administration, to K. H. McGhee, research engineer, Virginia Highway & Transportation Research Council, April 7, 1977.

9. Memorandum from F. T. Wagner, head, Materials and Test Unit, Department of Transportation, North Carolina to K. H. McGhee, research engineer, Virginia Highway & Transportation Research Council, April 8, 1977.

10. Memorandum from J. McChord, director of materials, Department of Transportation, Frankfort, Kentucky, to K. H. McGhee, research engineer, Virginia Highway & Transportation Research Council, April 4, 1977.

11. National Crushed Stone Association, "Guidelines for Construction of Serviceable Crushed Stone Bases," Washington, D. C., 1976.

12. Runkle, S. N., "Review of a Statistical Specification for Pugmill Mixed Material," Virginia Highway & Transportation Research Council, February 1974.